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# TRANSACTIONS

OF

# The Canadian Society of Civil Engineers.

VOL. XIV., PART I.

JANUARY TO JUNE,
1900.

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PRINTED FOR THE SOCIETY
BY JOHN LOVELL & SON.
1900.

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NOTE :—Owing to delay in the receipt of the Manuscript of Professor Owen's lectures on "Transmission of Electrical Power," the publication of Volume XIII, Part II, is held over.

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## INSTRUCTIONS FOR PREPARING PAPERS, ETC.

In writing papers, or discussions on papers, the use of the first person should be avoided.

They should be legibly written on foolscap paper, on one side only, with a margin on the left side.

Illustrations, when necessary, should be drawn on the dull side of tracing linen to as small a scale as is consistent with distinctness. They should not be more than 10 inches in height, and *in no case* should any one figure exceed this height. Black ink only should be used, and all lines, lettering, etc., must be clear and distinct.

When necessary to illustrate a paper for reading, diagrams must be furnished. These must be bold, distinct and clearly visible in detail for a distance of thirty feet.

Papers which have been read before other Societies, or have been published, cannot be read at meetings of this Society.

All communications must be forwarded to the Secretary of the Society, from whom any further information may be obtained.

The attention of Members is called to By-laws 46 and 47.

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TO 1901.

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Thursday, 18th January.

ROBERT A. ROSS, Member of Council, in the Chair.

Prof. R. B. Owens delivered the third of a series of lectures on the "Transmission of Electrical Power."—(Vol. XIII., Part II.)

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Thursday, 15th February.

DR. H. T. BOVEY, President, in the Chair.

The following donations to the Library were reported :—

Volumes of "Scientific American," from Mr. Geo. S. Brush, M.Can.Soc.C.E.

"A Text Book of Mineralogy," from Mr. J. Spenard, A.M.Can. Soc.C.E.

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*Paper No 148.*

## CONSTRUCTION OF THE ONTARIO AND BERRI STREET SUBWAYS, BROCK STREET TUNNEL, AND NOTRE DAME STREET VIADUCT.

By STUART HOWARD, M. Can. Soc. C. E.

The writer, in describing in a general way the construction of the several works in this Paper, wishes to state that the structures differ very little from works built for the same purpose, but hopes that the drawings and contemplated and final completed costs may be of some use to his brother Engineers. The writer, who was fortunate enough to have charge of the designing and superintendence of the above mentioned works, wishes to thank Mr. P. W. St. George, M. Can. Soc. C. E., under whose supervision they were carried out, for his courtesy in allowing him to write this paper.

Concrete was used very extensively in all the foundations below ground, the masonry being all rock-faced Ashlar in regular courses, the backing being made up of large stones and concrete, well-bonded to the face stones ; the cement was the best to be had in the market, and was mixed in the proportion of 1 of cement to 2 of coarse sand, for the Ashlar work and backing, and for the bridge seats and coping 1 to 1.

## ONTARIO STREET SUBWAY.

This subway was constructed to permit of a roadway being built under the C. P. R. tracks at Hochelaga, which at this point numbered ten in all. The masonry retaining walls extend upon the south side, from the east side of the street in front of the Stock Yards hotel to the west side of Moreau street, and on the north side, from Moreau street to a point about 22 feet west of the west end of the subway proper, the walls being for a considerable distance on solid rock. The descending grade at each end is 1 in 20. The bridge carrying the tracks being 165 feet in length, by 40 feet in width, with a clear headway of 12 feet, it was impossible to give any more height, as the lower portion of the subway could not have been properly drained. The total length of the retaining walls is as follows: upon the south side, 649 feet, and on the north, 378 feet, and are built according to the sections shown on plate 18, and stepped down so as to conform to the natural surface of the upper ground. The clear span between the perpendicular retaining walls is 40 feet; the copings being at the upper ends about 2 feet above the street surface and 16 feet at the subway, and surmounted by an iron fence composed of cast-iron posts placed every 10 feet, with 3 lines of 1 1-2 inch gas pipe, the fence being 3 feet 3 inches in height; this fence is returned across the bridge on the limit of the Railway Right-of-way, the posts being bolted to the trough plates, and to a plate holding in the concrete, in which the ties are bedded. The subway bridge is 156 feet long and 40 feet in width, with a central wall extending the entire length, on which are bolted 14 latticed channel columns, carrying plate girders of a depth of 2 feet, the ends of these girders resting upon the bridge seats of the abutment walls. At right angles to these plate-girders are bolted iron trough plates, the ties being bedded in concrete in these troughs, and brought to the proper railway grade. The concrete for this work was composed of 1 of Portland cement, 1 of clean river sand, and 4 parts of broken stone.

The excavation under the tracks was taken out without interrupting the railway traffic, piles having been driven close to the side of the rails in such a position as not to interfere with the masonry work: they were then cut off and capped, and stringers slipped in under the ties, the whole making a pile trestlework. The retaining wall at the west end was returned up the east side of the street in front of the Stock Yards hotel, a distance of 50 feet, the street being graded to give easy access to Ontario street. The subway from end to end has been paved in scoria blocks, 5 foot sidewalks laid, new drain put in, as also water and gas pipes, and gullies.

The estimated cost was $85,000, and the actual cost as per details:

| | | | | | |
|---|---|---|---|---|---|
| Earth excavation.. .. .. .. | 11,970 | cub.-yds. at | 29½ cents | = | $ 3,531.15 |
| Rock excavation.. .. .. .. | 52 | " | 80½ " | = | 41.86 |
| Rock faced Ashlar .. .. .. | 3,061 | " | $8.57½ | = | 26,248.07 |
| Cut stone Masonry.. .. .. | 200 | " | $10.25½ | = | 2,051.00 |
| Stone behind walls .. .. .. | 663 | " | $3.00 | = | 1,989.00 |
| Temporary trestle.. .. .. .. .. .. .. .. .. .. .. .. .. .. | | | | = | 3,500.00 |
| Timber in same—price allowed for old.. .. .. .. .. .. | | | | = | 1,215.00 |
| Paving.. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. | | | | = | 11,305.91 |
| Fencing .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. | | | | = | 1,200.00 |
| Concrete.. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. | | | | = | 5,000.00 |
| Iron superstructure and columns.. .. .. .. .. .. .. .. .. | | | | = | 16,400.00 |
| Superintendence and sewers.. .. .. .. .. .. .. .. .. .. .. | | | | = | 10,476.00 |
| | | | | | $83,557.99 |

The cost of the subway proper under the tracks, including excavation, masonry, temporary bridge, bridge and floor system, amounted to $38,000.00; the cubic contents measured to the outside of the masonry walls were 169,000 cubic feet, equalling 22½ cents per cubic foot; $243.00 per lineal foot of subway, or $4.50 per square foot of area.

The contractors for the ironwork were the Dominion Bridge Co., and for all the other items Messrs. Laurier, Rheaume & Desormeau.

## BROCK STREET TUNNEL.

This tunnel was constructed, to open up a roadway from Craig street to the wharf, and is situated at Beaudry street, about eight-tenths of a mile east of the Post Office, and a third of a mile east of the Berri street subway, and has already proved its great use, the carters being able to take double the load they were formerly able to, the approach from the wharf being so heavy and the descending grade so steep by the ramps to Notre Dame street and down again to Craig street. The total length of the tunnel and approaches is 905 feet, measured from the south side of Craig street to the southerly limit of the Canadian Pacific Railway's cribwork on the wharf; the tunnel itself from portal to portal has a length of 666½ feet with a grade of 1 in 43.

The approach at the Craig street end is 200 ft. long and 40 ft. wide, with a 10 foot sidewalk on the west side, leaving a clear roadway of 30 feet. Upon the east side of this approach is a masonry retaining wall, which sustains an upper roadway leading from Craig street to Notre Dame, of a width of 35 feet. This roadway has a grade of 1 in 14½, the height at the north portal being 21 feet above the tunnel floor. The coping of this wall is stepped to suit the grade of the upper street, with an iron pipe fence similar in construction to that at the Ontario street subway.

The portals of the tunnel are of masonry, and extend a length of 8 feet, the arch stones being toothed so that the courses of brickwork are built into them exactly. The arch is a semi-circle with a radius of 15 feet, and springs from the floor of the tunnel, thereby giving a clear headway at the key stone of 15 feet. The brickwork at the haunches immediately above the masonry springing course is 4 feet 3 inches wide, and is stepped up until at the key it is 1 foot 5 inches in depth.

The intrados of the arch is of fire brick, 9 inches in thickness, the backing being of hard red bricks, the whole being laid in cement mortar, 1 of cement to 2 of sand; this brickwork has been most carefully laid, and thoroughly bonded, and all the courses kept perfectly straight throughout. On the top of the brickwork, and extending nearly to the haunches, is a thickness of 9 inches of concrete, composed of 1 of cement, 2 of sand and 4 of stone, and over this concrete No. 20 galvanized corrugated iron; the stone filling to the underside of the tunnel roof planking being carefully laid in by hand and well consolidated. This stone filling acts as a drain, any water from above descending to the 6 inch open jointed pipe which is laid upon a bed of concrete, and on the same grade as the tunnel.

The north end of the tunnel for a distance of 103 feet was built in open cutting, as the depth over the crown would not admit of tunnelling, and the excavation did not interfere with any traffic. After the brickwork, concrete and iron were completed, the excavated material was filled over, and again brought to the original surface. This work, as also the north east retaining wall, was completed December 15th, 1893.

The south approach consists of a bridge to carry the tracks of the Canadian Pacific Railway; the abutments are 36 feet apart with a mean length of 39 feet, and, in order not to interrupt the traffic of the railway, the tracks had to be carried by a temporary trestlework. As at the Ontario street subway, piles were driven from above and close to the track; they were then cut off below the surface, and timber caps put on; stringers were then slipped under the ties, forming a regular timber trestlework; the excavation was then proceeded with, and cross bracing put in wherever necessary. The piles were driven in such a position as not to interfere with the building of the abutments. This mode of procedure was most satisfactory, the entire work being done very expeditiously. The tracks are carried upon eight plate girders, 2 feet 6 inches deep, put in in pairs and braced sideways by latticed girders; the flooring is composed of 10 inches x 10 inches creosoted timbers laid close together, the joints being thoroughly caulked with oakum; this structure cost $4,900.00. On the outside of the south portal hollow quoins were left in the recesses of the abutments, into which solid lock gates 12 inches thick and 14½ feet in height are fitted, the gates closing against a 12 inch

timber, let into the mitre wall, and well bolted; this timber is put in after the close of navigation, a 6 inch plank taking its place during the summer season. This gate was to prevent the entrance of the water during any rise in the river, but the water finds its way from behind the abutments, and up into the tunnel by its pressure, and is of little use except to prevent ice entering the tunnel and injuring the walls. The difference of level between the wharf and Craig street is 21 feet, the highest freshet known being 13½ feet above the floor of the tunnel, at the south end.

The drifting of the tunnel was accomplished without any accident whatever, and the timbers were placed as shown on the section. A heading was first drifted from the south end about 10 x 11 feet, and timbered; the excavation for the tunnel itself was then carefully done, the timbering being put in as soon as each section was excavated, until the whole area required was complete. The upper roof longitudinals as also the roof boarding were left in. When the masonry haunches were completed, the centering for the brickwork proper was put in, and the upright timbers cut out one by one as the work progressed, the roof timbers being supported from the brickwork. After the concrete over the arch was done, the corrugated iron was put in position and the space above filled in with large sized broken stone.

The tunnel was built from both ends, and when the walls met, no difference was found either in alignment or level. The key of the arch is of stone. When the walls met in working from each end, a shaft was sunk from the street above, and the brickwork completed, an inscription stone containing the city archives, coins of the realm, photographs, and plans in a copper box being placed in the side wall. In the tunnel there were used approximately 350,000 fire bricks, 1,300,000 hard red bricks, 1,450 cubic yards of masonry, 1,110 cubic yards of concrete, 50,000 pounds of corrugated iron, 3,600 cubic yards of stone filling, and 15,000 cubic yards of excavation. The north approach has been paved with scoria blocks, the tunnel and south approach with porphyry blocks. The roadway is 25 feet in width with a 5 foot elevated sidewalk on the west side only, protected by an iron pipe railing.

### *"Engineers' Estimate of Cost."*

| | | | |
|---|---|---|---|
| Excavation for walls and open built tunnel | c.ys. | 6,750 at 50c. | $3,375 00 |
| Filling over tunnel (open cut) | " | 2,200 " 25c. | 550 00 |
| Masonry in retaining walls | " | 900 " $9.00 | 8,100 00 |
| do in north and south portals | " | 390 " $12.00 | 4,680 00 |
| Cut stone copings, walls and portals | " | 80 " 14.00 | 1,120 00 |
| Excavation for Can. Pac. Ry. bridge | " | 1,650 " 1.00 | 1,650 00 |
| Pulling down and rebuilding old ramp wall | " | 145 " 5.00 | 725 00 |
| Stone behind walls | " | 250 " 2.50 | 625 00 |
| Temporary C.P.R. bridge | | | 1,500 00 |
| Open built tunnel | L. ft. | 96 x $126.00 = | $12,096 00 |

*Tunnel drifted* (per lin. foot).

| | | | |
|---|---|---|---|
| Excavation | c.ys. | 27 at $ 1.50 = | $40 50 |
| Key stone | " | .055 " 12.00 | 0 66 |
| Masonry foundation. | " | 1.54 " 10.00 | 15 40 |
| Fire bricks | in. | 600 " 40.00 | 24 00 |
| Red do | " | 2,000 " 24.00 | 48 00 |
| Concrete | cys. | 2 " 9.00 | 18 00 |
| Corrugated iron | lbs. | 85 " 10 00 | 8 50 |
| Stone behind wall | cys. | 1.7 " 3.00 | 5 10 |
| Tile pipe, 6" | 1 ft. | 2 " 0.35 | 0 70 |
| Centres and headings | " | 1 " 9.00 | 9 00 |
| Filling over arch | cys: | 4.25 " 1.00 | 4 25 |
| Extras | | | 0 89 |
| | | $176 00 x 557 feet = | 98,032 00 |
| | | | $132,453 00 |

The contractors' final estimate for this work amounted to $131,696 67.

The other items in the Engineers' Approximate Cost, not included in the contract for the tunnel proper, were as follows:—

| | | |
|---|---|---|
| Railings on Copings and in Tunnel, 942 lineal ft. at $1.75 | = | $ 1,648.00 |
| Iron Bridge to carry C. P. R. tracks | = | 5,000.00 |
| Flood gates at South Portal | = | 400.00 |
| Porphyry Paving in Tunnel 1,964 sq. yds. at $4.50 | = | 8,838.00 |
| Sidewalk in Tunnel 300 " " 3.50 | = | 1,050.00 |
| Curb in Tunnel 667 lin. feet at 70c. | = | 467.00 |
| Curb at North End 200 " " $1.00 | = | 200.00 |
| Scoria Block Paving 1,092 sq. yds. at 3.50 | = | 3,822.00 |
| Drains and Macadamizing | = | 4,200.00 |
| Engineering and Contingencies | = | 17,000.00 |
| | | $42,625.00 |

This work was completed for the sum of $42,304.00, the Engineer's estimate for the total work being $175,078.00; whereas the total cost when completed was $174,000.00. This amount really covered many little items which were unforeseen.

The contractors for the tunnel portion of the work were Messrs. Lafontaine & Lemoine, and the iron bridge was put in by the Dominion Bridge Co.

## BERRI STREET SUBWAY.

This subway was commenced at the end of May, 1893, and before the winter set in the whole work, with the exception of the paving, was completed. This subway was designed to give an easy approach from Craig to Commissioners' street, Berri Lane, or Barrack street being widened, and a new street opened up between St. Louis and Notre Dame streets. The subway proper extends from Champ de Mars street to the Canadian Pacific Railway freight shed, a total distance of 430 feet. The approach from Champ de Mars street has a descending grade of 1 in 39 to the arch under Notre Dame street a distance of 180 feet, the roadway under the arch and the eastern approach from Commissioners' street being practically level for a distance of 250 feet.

Between Champ de Mars and Notre Dame streets there is an upper roadway upon the south side of the subway, with a width of 20 feet, and upon the same side, extending from Notre Dame to Commissioners' street, a similar upper road of a width of 30 feet. Heavy retaining walls are built to sustain these roadways, the copings being stepped to suit the grades of the streets, and surmounted by an iron pipe fence similar to the other structures already described. The arch was originally designed with a clear span of 45 feet, as permission could not be obtained from the Hospice St. Charles to build the north westerly retaining wall upon their property, but, at a later period after nearly all the arch stones had been cut, and the centres made, permission by the Hospital authorities was granted and the span widened to 52 feet. Any engineer now viewing this arch would naturally say, what an extraordinary structure, an arch at one end with the abutment carried forward to carry an iron viaduct. Why was it not all carried out in iron? When the Berri Street Subway was designed, no plans had been made, nor was the idea broached of any extension of the Canadian Pacific Railway into the present Place Viger Station, but when the work was well under construction, the scheme matured in time to prevent the building of the north westerly retaining wall.

The arch centering was made very strong, the lagging only being carried down to the top of the third course above the springing; many centres are made so weak that the weight of the arch stones causes the haunches to settle, and the crown to spring up. Care was also taken to build the arch evenly from both sides, so as to distribute the weights carefully; all the stones being used were also loaded upon the centering, keeping it in place.

The change in the span of the arch necessitated a great deal of extra work; new centres had to be made, and about 8 feet in height of the northerly abutment torn down, and moved forward 7 feet. By referring to the drawing of the arch, you will see that it is formed by radii from seven centers, which were arranged so that the curve might conform as nearly as possible to a true ellipse. In order to utilize as many as possible of the old arch stones, only a few had to be thrown out, the same radii being kept, and a new central one of a larger radius or 46 feet 6 inches inserted, raising the key stone, only 6 inches. The arch stones were cut very truly to the radii, and the whole structure built most carefully, and when the centres were removed no settlement whatever could be measured. The section of the arch shows clearly the top crown line, which was formed of masonry whenever depth would allow, and concrete in the shallow portions, the whole being covered with a 3 inch thickness of asphalt, allowing any water to drop into the back stone filling, and out through the weepholes. The retaining walls were built according to the section shown, with a face batter of 1 in 12, the coping being laid on a perfectly straight line, and with an iron pipe fence on top. The old retaining wall on Barrack street which the writer had the pleasure of building some years formerly was demolished, the stone as far as possible being used in the new work, and the south end of the Dalhousie Square depot was underpinned, and reconstructed to correspond with the other faces of the building. An iron staircase was built opposite St. Paul street, for pedestrians to descend to the lower level or subway. The writer has given sections of two elliptical arches, one with a rise of 1-3rd of the span, the other rise being 1-4th of the span, together with a table showing, by letters. the spans, radii and dimensions, which he hopes will be of some practical use.

*Engineers' Estimate for Subway with 45 Foot span Arch.*

| | | | | | |
|---|---|---|---|---|---|
| Excavation.. .. .. .. .. .. .. ..cub. yds. | 25,000 | at | 45c. | = | $11,250.00 |
| Filling over arch.. .. .. .. .. " | 900 | " | 30c. | = | 270.00 |
| Masonry in old wall torn down and rebuilt.. .. .. .. .. .. " | 800 | " | $6.00 | = | 4,800.00 |
| Masonry in arch abutments.. .. " | 650 | " | 9.00 | = | 5,950.00 |
| Masonry in arch stones and copings | 267 | " | 14.00 | = | 3,738.00 |
| Masonry in retaining walls...cub. yds. | 3,410 | " | 9.00 | = | 30,690.00 |
| Asphalt on arch.. .. .. .. .. ..sq. yds. | 400 | " | 2.50 | = | 1,000.00 |
| Stone behind walls.. .. .. .. ..cub. yds. | 650 | " | 2.50 | = | 1,625.00 |
| Cutting down wall at depot... " | 360 | " | 4.00 | = | 1,440.00 |
| Paving in scoria.. .. .. .. .. ..sq. yds. | 3,637 | " | 3.50 | = | 12,730.00 |
| Engineering and contingencies.. .. .. .. .. .. .. .. .. .. | | | | | 10,500.00 |
| | | | | | $83,593.00 |

| | |
|---|---|
| Deduct north-west wall, 922 cubic yards at $9.00.. .. | 8,298.00 |
| | $75,695.00 |
| Add additional cost of altering to 52′ span.. .. .. .. | 2,000.00 |
| | $77,695.00 |

The 45 foot span according to these figures would amount to $280 per foot in depth of arch, and for a 52 foot span $310 per foot in depth of arch. The cost of the arch proper according to contractors' figures amounted to $281 per foot in depth. The total cost of the contractors' work, including paving, amounted to $67,000, and the engineering and contingencies the sum of $10,255, making a total of $77,255. The contractors for the arch, paving and walls were Messrs. Madore and Frechette.

## NOTRE DAME STREET VIADUCT.

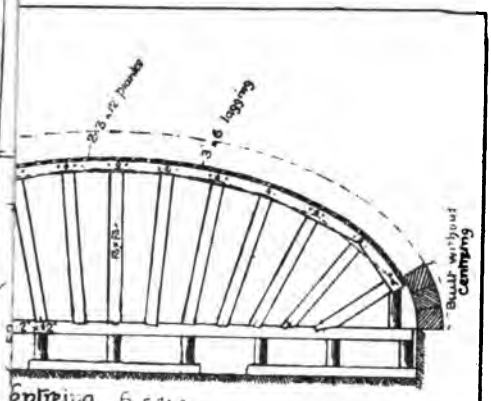
This viaduct has been built on the lines of Notre Dame street, between the Berri street subway arch and the East side of Lacroix street, for the accommodation of the Canadian Pacific Railway, their tracks leading to the new Viger Square Hotel Depot, passing beneath the structure. The northerly abutment of the Berri street arch had to be widened, not altogether for its additional strength, but in order to suit the spans of the bridge itself.

In addition to the two abutments, the northerly one of which was carried down Lacroix street on a curve, there are 21 pedestals with copings 3½ feet x 12 feet, and 22 with copings 3½ feet square. The pedestals as far as the north end of the old Dalhousie Square passenger depot are at right angles to the axis of the bridge, the remainder as far as the north abutment being placed at an angle of 54° 41′ 30″, so as to suit the curves of the tracks approaching the depot at Viger square the north abutment being parallel to them. The total length of the bridge proper between the abutment bridge seats being on the west side, 437′ 3″; on the centre line, 452′ 10″ and on the east side, 468′ 5″. The width of the bridge between the outside plates is 52′ 3″, the five lines of columns being placed 11 feet centre to centre. The columns are composed of 2 channel irons, and one eye beam, resting upon rocker bases, having also rockers at the top, to allow for the contraction and expansion of the steel. Upon these columns which are thoroughly braced transversely are placed the longitudinal plate girders. The spans of these girders are, 4 clear spans of 38′ 3″, 2 of 50′ 2½″, one of 40′ 2½″, and one of 59′ 6″, and one 34′ 1¼″, besides the tower spans. The columns

on the long pedestals are placed 7′ 9½″ centre to centre, which constitute the towers, the depth of the girders being 4′ 3″ deep for the short 38′ 3″ spans, and 5′ 3″ for the longer ones.

On the top these girders and at right angles to them are placed the iron floor beams, composed of 10¼″ eye-beams, extending the whole width of the bridge, and at 4 feet centres; upon these eye-beams, which are bolted to the girders, are placed the 3-16 inch buckled plates, the buckled portions being 4 ft. sq., and made in long lengths; these plates are riveted to the eye-beams and strengthened longitudinally by T bars. The roadway is composed of concrete and wooden block paving, with a stone curb, and firimite sidewalk of a width of 8′ 1½″ upon each side, leaving a clear roadway between the curbs of 36 feet, a double electric car system of tracks being placed in the centre, the trolley poles which are of iron being bolted to the outside plates and bracketted to the columns. The buckled plates before the concrete was laid were painted with hot asphalt, which has had the desired effect of making the bridge almost perfectly water tight. The fence posts were of cast-iron 3 inches in diameter, with bosses; they were placed every 8 feet, and bolted to the outside plates; there are 4 lines of 1½ inch gas-pipes passing through them, the top bar being 3′ 4″ above the sidewalk level at the outside of the bridge. The triangular portion of the bridge down Lacroix street has a length of curved abutment of 140 feet. And on the south side one column and 2-60′ girder spans of a depth of 6 feet, girders extend from these outside girders to the bridge seat of the abutment, of a depth of 3 feet, and at distances of 13′ 3″ parallel to the main bridge. At right angles to them are placed 12 inch eye-beams, at 4 feet centres; and on these the buckled plates and scoria block paving with concrete. During the construction of the bridge a temporary roadway was made upon trestlework, west of the structure, so that the traffic was not much interfered with, the cars being carried this way, and only stopped when the connections were made at the north end. The old wall along the east side of Notre Dame street, which the writer also constructed for the Canadian Pacific Railway, had to be torn down, a great quantity of the stone being used in the bridge abutments and pedestals.

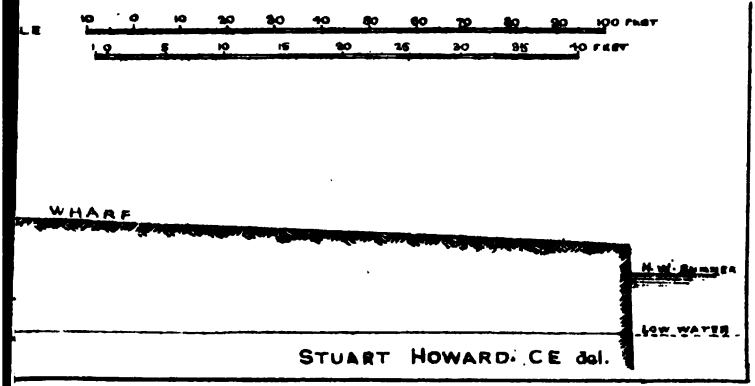
It would be well to note here that this wall was built during the winter months, the thermometer being from 15° to 20° below zero for days together, the sand and cement were warmed, as also the water, and the stones subjected to a steam jet to thoroughly clear-off any snow or ice, and yet we had great difficulty in pulling this wall apart, showing conclusively that masonry can, without any fear, be built during winter time, care being taken.

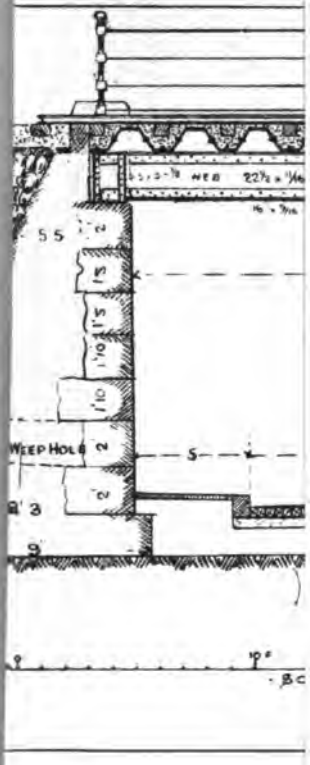



# STREET SUBWAY.

Montreal Dec. 1899.
Stuart Howard. C.E.
del:

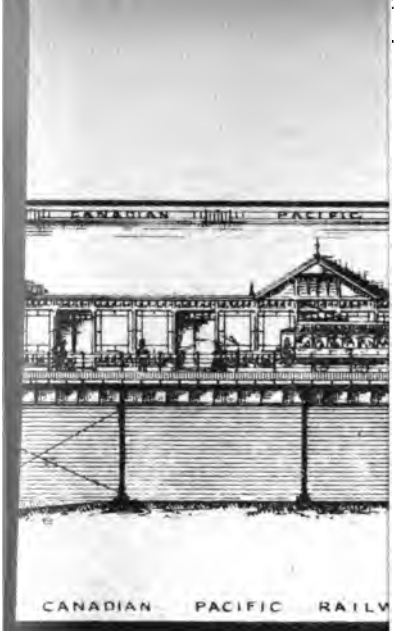
TRANSACTIONS CAN. SOC. C. E.
VOL. XIII PLATE XVII

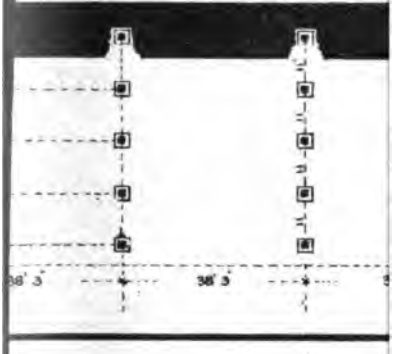


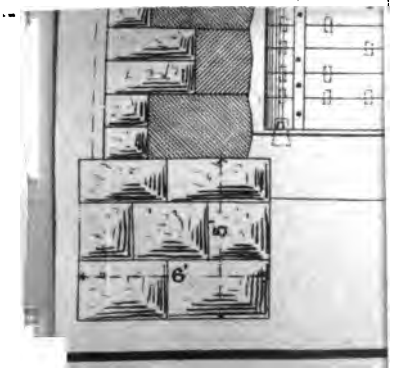

WEEP HOLE

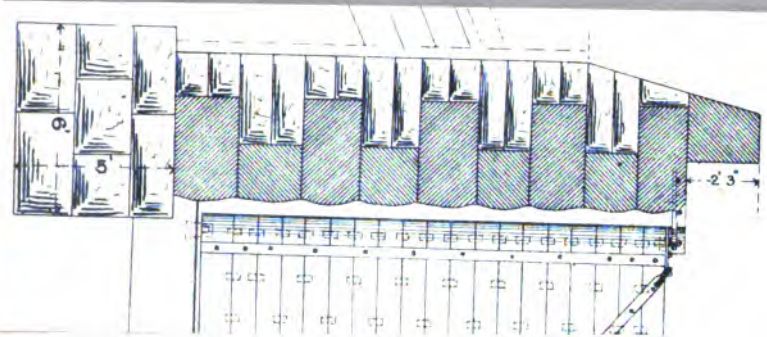
6'

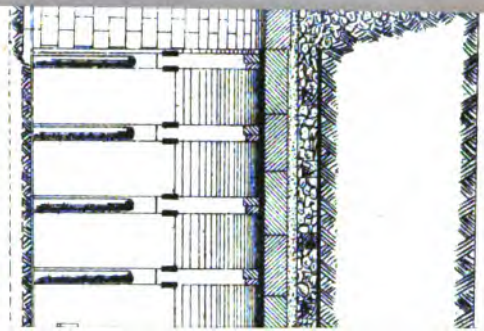
STUART HOWARD C.


CANADIAN PACIFIC


CANADIAN PACIFIC RAILV
38' 3"
38' 3"

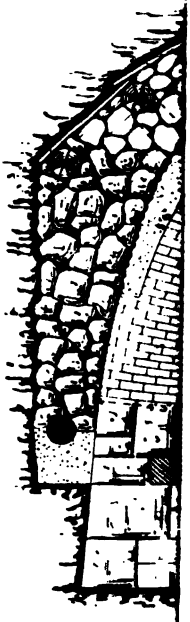

6'
10'

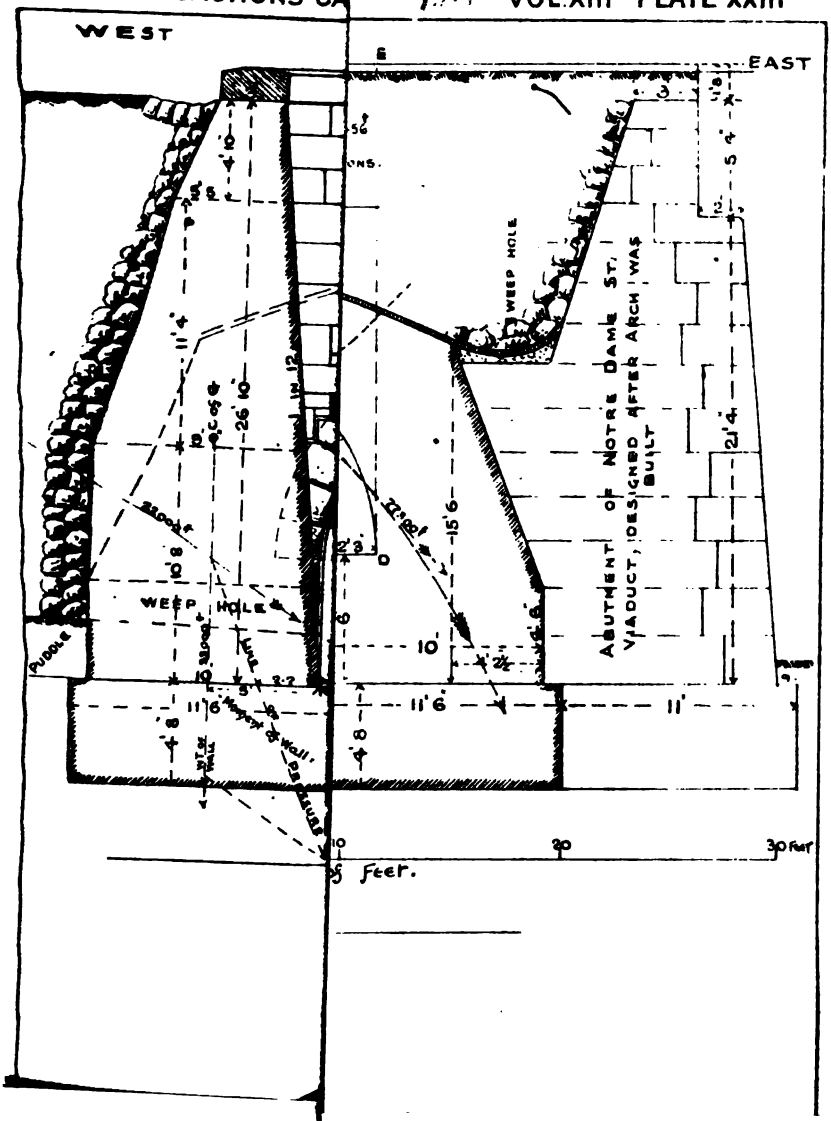

6'
5'
2' 3"



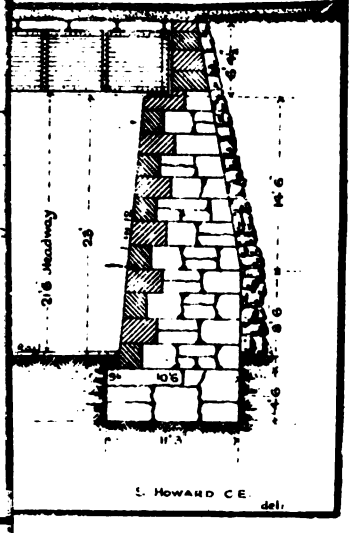
44 - 7

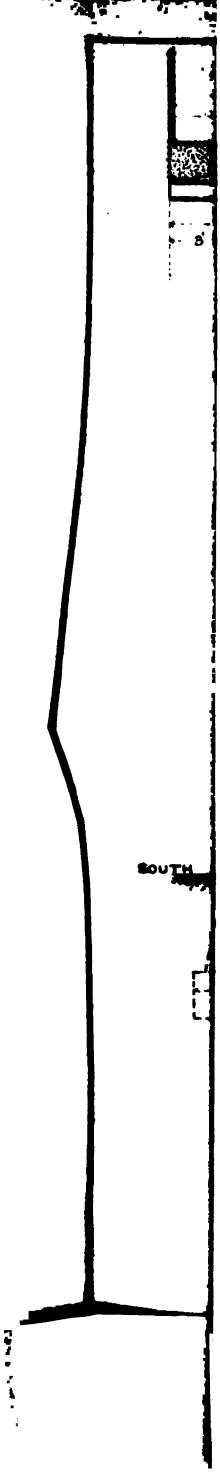





WEST
EAST
WEEP HOLE
ABUTMENT OF NOTRE DAME ST. VIAUDUCT, DESIGNED AFTER ARCH WAS BUILT
WEEP HOLE
PUDDLE
11'6"
11'6"
11'
10
20
30 Feet.

Section of East Abutment.




SOUTH

*Engineers' Estimate of Cost.*

| | | | |
|---|---|---|---|
| Excavation.. .. .. .. .. .. ..cub. yds. | 28,000 | $ 0.65 | $18,200.00 |
| Tearing down old walls.. .. .. .. " | 1,300 | 1.50 | 1,950.00 |
| Temporary bridge and grading road.. .. .. .. .. .. .. .. .. .. | | | 3,000.00 |
| Masonry in abutments.. .. .. ..cub. yds. | 1,400 | 9.00 | 12,600.00 |
| Paving and sidewalks.. .. .. ..sq. yds. | 2,720 | 3.50 | 9,520.00 |
| Masonry in pedestals.. .. .. ..cub. yds. | 414 | 10.00 | 4,140.00 |
| Cutstone on pedestals and bridge abutments.. .. .. .. .. .. .. " | 180 | 14.00 | 2,520.00 |
| Removal, etc., of water-pipes.... | | | 2,500.00 |
| Stone-filling behind walls.. ..cub. yds. | 100 | 4.00 | 400.00 |
| Superintendence and contingencies | | | 15,000.00 |
| | | | $69,830.00 |

*Iron Superstructure.*

In columns.. .. .. .. ..110,000 lbs.
In girders.. .. .. .. .. ..460,000 "
Buckled plates & T beams 250,000 "
Floor beams.. .. .. .. ..165,000 "
In cross bracing, etc.. .. 97,000 " 1,082,000 lbs. at 3½ cts., $35,665 which equals a total cost of $105,495.00. The amounts paid were to the contractors, Madore & Frechette, $43,270.00; to the Dominion Bridge Co., $35,505.00; for blocks for roadway alteration of water pipes, superintendence and small contingencies, $24,191,000, or a total of $102,966.00; the amount of iron in the Bridge Co.'s tender was 1,090,000 pounds.

The Lacroix street portion was estimated at $25,000.00; and the amounts paid were to the contractors, Madore & Frechette, $16,521.00; to the Dominion Bridge Co., $6,568.00, and for superintendence, $1,600.00, a sum total of $24,689.00.

Taking out the items in the bridge proper, namely, pedestals, steelwork and flooring, the structure being to back of bridge seats, a mean of 460 feet in length, a width of 52 feet, and an average depth of 30 feet, the cubic contents would be 717,600 cubic feet, at a cost of $40,425.00; or 5.63 cents per cubic foot, $88.00 per lineal foot of bridge, or $1.70 per square foot of area. The columns are 22 feet long, and any extra depth can be allowed up to 30 foot columns by adding $1.25 per cubic foot to the above cost.

NOTE.—The plates in connection with this paper (following) have been incorrectly engraved as belonging to Vol. XIII.

Thursday, 15th March.

DR. H. T. BOVEY, President, in the Chair.

Mr. L. Skaife gave notice of his intention to bring forward the following motion: "That the rooms of the Society be kept open on all public holidays."

A short paper on "The Canadian Pacific Railway Transfer Slip," by Mr. H. J. Wadsworth Finch, Stud.Can.Soc.C.E., was read.

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Thursday, 29th March.

DR. H. T. BOVEY, President, in the Chair.

Prof. R. B. Owens delivered the fourth, and last of a course of lectures on the "Transmission of Electrical Power."—(Vol. XIII., Part II.)

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Thursday, 12th April.

DR. H. T. BOVEY, President, in the Chair.

The following donation to the Library was announced:—

"Architectural Engineering," from Mr. Paul Weatherbe, A.M.Can. Soc.C.E.

Mr. L. Skaife, seconded by Mr. T. W. Lesage, moved: "That the rooms of the Society be kept open on all public holidays."

Moved in amendment by Mr. G. Janin, "That the matter be referred to the Library Committee, with the addition, that the Society thinks that it is not necessary to keep the rooms open on holidays."

After some discussion, Mr. Skaife withdrew his motion, and the matter was allowed to drop.

Messrs. J. S. Vindin, L. Skaife and R. T. Gough, having been appointed scrutineers of the ballot for the election of members, declared the following elected:—

MEMBERS.

C. H. DAVIS, W. A. JOHNSON,
J. G. SULLIVAN.

ASSOCIATE MEMBERS.

E. A. FORWARD, B. H. FRASER,
H. B. WALKEM.

Transferred from the class of Associate Member to the class of Member:

R. S. LEA.

Transferred from the class of Student to the class of Associate Member:

R. E. HUNTER.

STUDENTS.

H. M. DAVY, A. J. R. MACDOUGAL,
A. L. KILLALY, R. T. MACKEEN.
A. C. MACDOUGALL, J. P. WALKER,
F. A. WISE.

*Paper No. 149.*

# PURIFICATION OF SEWAGE BY MEANS OF THE SOIL

(Sewage Farm).

By GEO. JANIN, A.M.CAN.SOC.C.E.

## PROTECTION OF WATER SUPPLIES BY THE SUPPRESSION OF THE RIVERS' POLLUTION.

In mentioning the danger of polluting by the sewage of the cities the streams which generally supply drinkable water to these same cities or their surroundings, we must give credit to Great Britain for having first submitted to her scientists the study of this great question, which brought on, in 1878, the vote, by the British Parliament, of the "Rivers' Pollution Prevention Act," obliging the municipalities to practically eliminate the noxious principles of sewage water before emptying it in the streams.

England's example was followed in the whole of Europe, especially in Paris, where the "Roads and Bridges Engineers Corps," to which I had the honour to belong, has made the most marvelous progress in the establishment of purification fields at Gennevilliers and Achères.

Unfortunately, in this country, we do not follow this example, and, boasting of the ampleness and copiousness of our rivers, our municipalities seem to court epidemics, in spite of the advices of their Boards of Health, and empty their impure sewage in the streams at the bottom of which generally lie deposits or banks of sawdust from the numerous mills lining the shores.

This neglect of the interests of health arises principally from the considerable expenditure anticipated by engineers in estimates based on special methods of treatment for sewage water. If, then, the most practical and cheapest system of sewage farms continued without any serious enquiry to be declared unsatisfactory and unavailable in our climate, there would be a danger of sewage pollution of streams becoming permanent in this country, to the great detriment of the purity of our water supplies generally.

In order to assist in counteracting this state of things, the author of this paper will endeavour to destroy any prejudice which may still exist against sewage farms amongst the members of the profession.

## PURIFICATION BY THE SOIL AND AGRICULTURAL UTILIZATION OF SEWAGE.

Soil purification of sewage is the method now in favour in England, Germany and France. In the last named country, I had the

honour of co-operating in the installation of this system with the learned engineer Durand Claye and his colleague, M. Masson, inspector of the drainage of the City of Paris. This process consists, as you know, in the filtration of sewage water through a permeable soil affording a sufficient thickness and all facilities for the outflow of the purified waters, either by a sufficient porosity of the subjacent stratum or by artificial drainage.

That the soil is the most perfect purifier of waters charged with organic matter is proved, first, by the organic purity of spring water, second, by careful experiments with impure waters, which are subjected to analysis before and after percolation through the soil.

THEORY OF THE PURIFICATION OF SEWAGE POURED ON THE SOIL.

The following information is taken principally from the remarkable report of Mr. Schloesing, reporter of the Health Committee of Paris (France).

When impure water, that of sewers for instance, is spread on light land, the insoluble matter remains on the surface. Some of the more minute portions, however, go a short way down, but soon stop; so that below a depth which varies with the nature of the ground, but which seems never to exceed two or three inches, there are no more solid particles to be found. This is the first effect produced; it is simply a mechanical filtration.

The water, now free from insoluble matter, goes down further, and an extremely thin coating of liquid gathers round each particle of earth. Thus divided, the water offers an enormous surface to the air confined in the ground; and the second effect of irrigation then takes place, *i.e.*, the combustion, by the oxygen in the air, of the organic matter dissolved in the sewage water. This is not a violent and visible phenomenon like fire; it is a slow combustion without any external sign, and which reduces all organic impurities into carbonic acid, water and nitric acid. This combustion is even more perfect than the other, for it burns up the nitrogen, which fire cannot do.

As to the insoluble matter retained on the surface, it also undergoes slow combustion, especially when ploughing has divided it and incorporated it with the soil. All that remains is a fine sand which will constitute a part of the minerals in the ground.

But how is it then that the oxygen of the air can under these conditions completely consume the organic matter of sewage and nitrify the nitrogen?

According to the researches of Messrs. Schloesing and Muntz, in in 1877-1878, this remarkable property is owing to the action of living organisms, existing in vegetable soil as well as in sewage water, and able, like the microderma aceti and others, the functions of which Pasteur has so well described, to transport the oxygen of the air on organic matter.

These are the experiments of Mr. Schloesing: If you pour regularly and in small doses sewage water on any kind of sand, mixed with a little humus and placed in a vertical glass tube, you will immediately get in the lower part perfectly pure water containing all the nitrogen of the sewage water in the form of nitric acid or nitrate, according to the nature of the sand.

If for this mixture of sand and humus or arable soil you substitute quartz sand calcined up to red heat, the purification does not take place in the first days, but only after a few weeks, and then, however, as completely as in the first case. The water collected in the lower part of the glass tube is perfectly pure; it contains all the nitrogen of the sewage water under the form of nitric acid.

Further, if, in the second case, and whilst complete purification is taking place, you send through the glass tube a current of chloroform (experiments of Mr. Muntz), the purifying power ceases completely and resumes only after the sand has undergone a protracted washing.

From these experiments we may draw the following conclusions:—

1. That purification by means of sand, free from organic matter, does not take place in the first days of the watering, because the germs of the nitrifying organisms are then non-existent.

2. That these germs can be brought in by sewage water and develop in sufficient quantities within a few weeks, and

3. That in vegetable soil, on the contrary, the purification begins immediately because the organisms are in full possession of the ground.

The researches of Mr. Muntz indirectly confirm this explanation, for, as chloroform has the property of paralysing all organisms acting as ferments, the purifying power ceases as long as the action of chloroform lasts.

Let us now see, with regard to practical work, what conclusions we can draw from these scientific notions, to be applied to the choice of land intended for the purification of sewage water. As the action of air is indispensable and the water should get through the soil easily, the latter should as much as possible be light, permeable and be of sufficient thickness above the sheet of liquid and the impermeable stratum. The filtered water should also be able to flow away easily either because the surface of the impermeable stratum offers an easy grade, or else artificially by means of drains. It is absolutely necessary to get rid of the filtered water. It is easy to understand that, if it could accumulate in the same place, the thickness of purifying soil might be diminished and become too slight to secure complete purification.

More or less arable ground, therefore, containing a greater or a smaller proportion of humus, is suitable for purifying sewage water,

whilst assimilating its fertilizing elements. But the poorest land, pure sand even, secures after a short time a purification just as complete, as the sewage water itself furnishes the germs of the same nitrifying organisms as those enclosed in the humus of arable land.

In many documents relative to sewage water, plants are erroneously classed with the soil as purifying agents; the bare soil without any vegetation suffices for complete purification.

Plants live on mineral compounds: carbonic acid, water, ammonia, nitric acid, phosphates, etc., and do not absorb organic substances, or only very small proportions of them.

However, they help purification, but in another way, mechanically as it were. They facilitate and determine in a notable degree the evaporation of the water poured on the ground, and thus contribute to getting rid of the liquids (experiments of Marié-Davy in the municipal garden of Gennevilliers). Besides, they consume a large portion of the ammonia or nitric acid produced, thus removing it from the purified water.

Therefore, as far as purification strictly speaking is concerned, cultivation is only of secondary importance; but from the agricultural standpoint this importance is very great, since we can thus make use of all the fertilizing elements contained in sewage water. This is the experiment of Mr. Marié-Davy: Four old stone-work tanks, 33 feet by 26, which had formerly been used for attempts at chemical purification, were filled to a height of 6 1-2 feet, with the average soil of the Gennevilliers' plain. Drainage pipes were placed at the bottom, which like the sides were covered with an impermeable coating. This impure water poured into each tank and the purified water issuing from the drainage pipes were both exactly measured. Different plants such as lucerne, maize, beet roots, corn, etc., were cultivated on the surface of the tanks and weighed with great care at harvest times.

Out of 12,700 cubic yards of liquid introduced per acre in half a year, less than 848 reached a depth of six feet. Generally speaking, Mr. Marié-Davy found that only one-fifteenth of the water poured on the surface reached the lower liquid level.

Vegetation acts therefore as a powerful vertical drainer; the soil transforms the impure water received, oxydizes it and produces an excellent liquid manure.

Plants in their turn absorb for their own benefit the useful elements of this manure, and give back to the atmosphere by means of evaporation the greater portion of the liquid which has served as a means of conveyance.

The soil is itself highly enriched by the organic matter which it takes from the sewage, and thus an economical value is given to the method of soil purification. Many analyses of vegetables grown on such soil have been made, and in no case has any diseased or unsafe

food product resulted from the use of these fields as market gardens.

Thus, the filtration through the soil and the agricultural utilisation complete one another. Not only purification, but also restoration, accomplish themselves, and the system bears out the axiom of the learned English hygienist, Edwin Chadwick: circulation always; stagnation, never!

In order to demonstrate the harmlessness of this system, it is advisable to quote the opinions and the experiments of a great many scientists, and first to mention the statement made by Dr. Frankland of the Royal Society who declares that: "Sewage, even though infected with cholera and typhoid fever, has never, when employed in irrigation, communicated disease either to parties living on the land so irrigated or to the consumers of the products thereof."

Here are now some extracts from a remarkable report made to the French Senate in December, 1888, by Mr. Cornil* in a debate concerning the purification of sewage water in Paris:—

"There has been much attention directed lately as to what becomes of micro-organisms when they pass through permeable soil. With my assistants, MM. Chantemesse and Widal, I have made—and Mr. Grancher on his side has also made—experiments to know what becomes of micro-organisms of a determined kind, easily recognized by its characteristics, poured on the surface of one of these large tubes filled with Gennevilliers soil.

"We have thus placed on the surface of our tubes microbes of typhoid fever, and we have sought if these micro-organisms of typhoid fever passed through the soil and were found in the water collected at the lower part of the tubes. Well, none pass through, even if the experiment is repeated for several weeks. Mr. Grancher repeated it for several consecutive months, and was unable to find even one.

"The soil at a given depth is therefore absolutely impermeable to microbes. This fact has also been confirmed by a series of experiments of another kind.

"When, by means of perforating apparatus, you obtain a sample of earth collected at a depth of one meter and examine it with reference to micro-organisms, none are found. You cannot find any of those which have been sown on the surface of the soil. Therefore, the microbes do not pass into the water of the drains which discharge the liquid poured as sewage water on the surface.

"After having filtered through a meter or a meter and a half of earth, this impure water appears perfectly clean, clear, containing scarcely any micro-organisms, and notably none of those sown on the surface. Thus, if you sow micro-organisms on the surface of

* Senator and Professor of Microbiology at the Faculty of Medicine of Paris.

soil and collect what issues from the test-tubes, *i.e.*, the drainage pipes, no trace is found of the micro-organisms sown; they have remained on the way, at 30, 40, 50 centimeters below the surface of the ground.

"They are not destroyed, but they are buried so to speak, and the soil is their cemetery.

"In any case, these experiments in tubes are absolutely conclusive with regard to the clarification of the water.

"These same experiments were made this year again in tanks at Gennevilliers, which had been used for chemical purification. These tanks are closed, in their lower portion, by an impermeable concrete; they are therefore absolutely watertight and filled with Gennevilliers soil.

"They have a surface of five acres, and a depth similar to that of the soil of the place, *i.e.*, two meters or two meters and a half.

"Sewage water was poured on this soil in considerable quantities for a certain number of days, or even for several months; then the water which had passed to the lower end was successively collected and found to be absolutely pure.

"It was examined in a chemical and bacteriological point of view.

"Chemically it contains very little dissolved matter.

"The quantity of micro-organisms contained in this water varies between 40 and a maximum of 600 per cubic centimeter, whilst that found in sewage water is 80,000 to 200,000 per cubic centimeter.

"This water issuing from the Gennevilliers soil does not contain any more micro-organisms than that of the Vanne or the Dhuys*. This results from all the experiments, all the analyses, and specially those made by M. Miquel. There are no more micro-organisms in this water than in the pure spring water furnished to the inhabitants of Paris."

After having heard the favourable opinion of so many learned men, after having passed through so many successful experiments made in the numerous purification fields of England, France and Germany, how is it that this system needs still to be defended against certain prejudices?

For it must be admitted that despite its superiority it has adversaries. Some are in good faith, but have been misled by imperfect experiments, to which reference will be made later. Others are mere grumblers blindly following their preferences for other systems or directly interested in the working of rival schemes.

Notwithstanding the evident superiority of the soil purification system, it is proper, considering the prejudices still existing, to illustrate, even briefly, the rules regulating the establishment and working of the purification fields—the violation of which rules has caused the few failures of the system.

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* Rivers supplying water to the City of Paris.

The purification of sewage by the soil can be accomplished in two different ways:

1st. The simple pouring or the flooding of prairies and heavy crops.

2nd. The methodical watering in succession of lands used for intensive gardening.

The largest number of failures recorded can be ascribed to the first method often used in England. For, inspired by a pennywise economy, in order to devote to the pouring the smallest possible surface, this method causing flows of high graded sewage water, cannot easily agree with the rules of the methodical filtering of impure water through the soil. The result frequently is an imperfect purification, and, in any case, a swampy aspect of the flooded fields as well as disagreeable emanations for the neighbourhood.

On the contrary, the following rules of the second method are always easy to observe; and their fulfilment is, in nearly all cases, crowned with success.

### OPERATION OF THE PURIFICATION THROUGH THE SOIL, WITH THE RULES ORIGINATING FROM SUCH OPERATION.

The operation process of purification through the soil admits of two motions, one of the water, the other of the air.

The motion of the water consists of:—

1st. The apportionment of impure waters on the surface of the ground, the filtering through the purifying soil, followed by the combustion of organic matters.

2nd. The outflowing of purified waters.

The motion of the air consists of an exchange between the soil and the atmosphere continuously renewing the oxygenic supply of the soil in proportion as such supply is spent by the combustion of organic matters contained in the water.

Man has an entire control of the motion of water and must regulate its pouring and outflowing according to the purifying power of the soil.

There is evidently a close connection between those two motions and the purifying power of the soil; to wit: the aeration and the movement of the water are really the purveyors whilst the soil is the instrument of the purification.

To sum up, the rules which govern a good and complete purification are as follows:—

First rule. To select ground porous enough to allow water to easily permeate and to give free access to a sufficient quantity of air for the working out of combustion. This degree of porousness or purifying power is determined by direct experiments, the best known being that of Dr. Frankland, which is as follows:—

In a vertical tube 2 meters in length and of $0^m 25$ to $0^m 30$ diameter, the lower end of which rests on gravel contained in a tank, samples of the different beds of the soil to be tested and placed in the same order in which they exist in the natural ground.

You pour daily on the soil a known and constant volume of sewage w[illegible] during several weeks. You then pass to a higher daily dose of sewage which you again keep up for several weeks, and so on, increasing the dose constantly until the analysis of the filtered liquids shows that the maximum dose has been reached, beyond which purification is no longer complete.

From this experiment it is easy to conclude what a square meter of the soil tested, and, consequently, what a hectare of it will purify at most per day or per year. We can only deduce from it the time required for purification, *i.e.*, the length of time during which the impure water should remain in the soil before being discharged. It is in days the quotient of the number of litres of water which the soil can hold* by the number of litres of impure water this same soil can purify per day.

By this method Frankland has shown that a cubic meter of pure sand or chalk can purify per day up to 33 litres of London sewage (litre—61.017 cubic inches), and that soil of mixed sand and clay or marl possesses still greater purifying power.

Evidently, in working out these data, we must remember that in practical work on a large scale we cannot realize exactly the conditions it is easy to observe in a small experiment, and that it is therefore desirable to reduce somewhat the maximum doses determined in the laboratory.

Second rule. To regulate the time of pouring and the quantity of sewage water poured each time, so that the water may take to run through the filtering soil *all* the time necessary for its purification.

Third rule. To underdrain, if necessary, in order to give to the purified water a regular issue.

In a word, the object is to obtain, as nearly as possible, a continuous and regular distribution since purification, a phenomenon of slow combustion and aeration—a mechanical fact—are both regular and continuous.

These are, as briefly enunciated as possible, the laws of operation of the purification by arable soil.

It follows from this exposé, as said before, that high dosed pouring alone has caused partial failure; not because it is bad in itself, but because the rules of purification by the soil imply certain

* A number which can easily be experimentally determined by weighing the tube full of dry earth before introducing the water and weighing it again after saturating contents and allowing discharge of liquid surplus.

absolute conditions such as selection of the ground, apportionment of sewage waters, underdraining, which are sometimes neglected on account of difficulties of ascertainment and of the great expenses they entail.

On the contrary, in all cases where irrigation has been moderately practised and where vegetation has helped purification, by returning to the atmosphere a portion of the purified water, success has been attained. To recall only a few of these successful experiments, I shall quote the purification field of Gennevilliers (Paris), where the annual dose of irrigation amounts to from one to four million gallons per acre; of Reims (France), where the dose is about the same; of Dantzig (Northern Prussia), where the average annual dose is three million gallons; and of Aldershot Camp (England), where the dose is also of about three million gallons per acre. A recent report (1899), made by Her Majesty's order to the Imperial Parliament, by Professor Fred. W. Andrews, M.D., F.R.C.P., D.P.H., etc., on the working of this sewage farm, the products of which are principally used for dairy purposes, affirms the absolute innocuousness of the system both with regard to odor and to the quality of the milk and butter produced by the animals fed on the yield of the farm. Professor Andrews says, amongst other things:—

"In concluding these remarks upon the farm, I will say that it "appeared to me to be an excellently managed one (by Col. Alf. "Jones, V.C., Assoc. M. Inst. C. E.), and as free from objection-"able features as any sewage farm could be. I could detect no "nuisance at the time of my visit . . . . . .
"I am unable to find any conditions on the farm liable to render "the milk produced thereon more dangerous for human consumption "than that from any other well-managed sewage farm, and I have "already stated that no evil results have yet been proved to occur "from sewage fed dairy produce as a whole."

These purification fields not only fill the hygienic programme of complete purification, harmlessness, absence of perceptible emanation for the neighbourhood, but also fill the economical programme as in Gennevilliers, where the value of land has increased fourfold since it is irrigated; in Reims where a company has rented the purification field for a period of thirty-six years, and after paying a heavy annual rental to the city, still realizes large benefits, and in Dantzig, where the purification field is rented at large profits.

The testimony in favour of the system with regard to its already lengthy applications in Europe and the more recent ones in the United States could be multiplied, but it will be preferable not to go beyond the instances already mentioned and to discuss the last and principal objection of the adversaries of the system, *i.e.*, that the severity of Canadian winters would interrupt the work of purification.

Like at Dantzig, where the thermometer registers as many degrees of cold as here, and where the system has been operating uninterruptedly for twenty-five years, it has been working successfully for three years in spite of the Canadian winters at only a few miles from Montreal. The sewage water of St. Laurent College has been successfully undergoing purification since January, 1896. The following extract from the annual report for 1897 of the Board of Health of the Province of Quebec confirms this, and also gives a description of the first sewage farm established in the province. Plan is annexed. The extract is as follows:—

"SEWAGE FARM OF ST. LAURENT COLLEGE.

"As we foretold in our last annual report, we can this year speak "by experience of sewage farms in our climate.

"Effectively the St. Laurent College, near Montreal, which dis- "charged its sewage water in a small creek passing through the "village, entrusted in the autumn of 1897 the establishment of a "farm for distribution and purification of its sewage to Mr. G. "Janin, a civil engineer of Montreal, who had formerly taken part "in the sanitation works of Paris.

"The works of this farm, which we followed with attention, were "finished sufficiently early in the winter, and of January, 1898, to "enable us to state that, as we had supposed, the methodical dis- "tribution of the sewage had gone on with ease in temperature "of more than 20 degrees Fahrenheit below zero. As there is no "reason for supposing that what took place in the severe winter "of last year will not equally take place in subsequent winters, "we can conclude that it is now proved that sewage farms are "perfectly compatible with the climate of the Province of Quebec.

"The sewage farm at St. Laurent College occupies a surface of "an acre and a half, and receives at present the sewage water of a "population of about five hundred persons, or an average flow "of 4,500 gallons of sewage water per 24 hours. This sewage, "which formerly was discharged directly into the village ditch, "now passes through a 10-inch eathenware pipe into a 9.000 "gallon tank placed beside the building containing the motive "power used for the different requirements of the College.

"This tank is absolutely watertight, and can, in case of need, "contain all the sewage water produced in 48 hours. A centri- "fugal pump placed in a well beside the tank, and worked by the "ordinary college motive power, raises in about 40 minutes the "daily amount of sewage, and forces it through an iron duct three "inches in diameter, into a distribution canal running along the "south side of the farm. In order that, after the pumping has "stopped working, nothing of a nature to obstruct may remain "in the elevating pipe, it is emptied into the tank as soon as the "pumping is over.

"From the distribution canal the sewage is directed at will by "means of gates into any one of the twelve longitudinal and five "lateral trenches which divide the farm into fifty-five parallelo-"grams of fifty feet in length, and twenty feet in width each, on "which vegetables, fertilized and watered by the sewage, are cul-"tivated. In the middle of each of these parallelograms there has "been placed porous earthenware pipes of three inches diameter, "about four feet below the surface of the ground. This drain "collects the sewage water after it has undergone purification in "the soil*, and conveys it to a ditch running along the northern "side of the farm. This ditch ends in the village creek, into which "the unpurified sewage of the college formerly flowed.

"Around the farm, large ditches have been opened or regraded "in order to let the water of the neighbouring ground discharge "freely and to prevent it from invading the farm or overloading "the underground sheet of water below said farm.

"To hinder the inconvenience resulting from the proximity of "the underground sheet of water, which in this low ground is only "a few feet below the surface, the level of the ground has been "raised by a filling of arable land, from sixteen to eighteen inches "in height.

"Not only has the sewage farm worked uninterruptedly in winter, "as we have said previously, but during the summer the produce "has been very satisfactory although no great efforts were made "for cultivation.

"Finally, then, we must note the success of this first application "in our climate of a system which works to the advantage of health "in the other countries of the old and new continents, and direct "to it the attention of our municipalities."

The members of the profession will find the system tried both more completely and on a larger scale in the sewage farm of St. Denis ward of this City. The works which have been entrusted to the writer by the City Corporation (under the direction of Mr. P. W. St. George, city surveyor, and Mr. S. Howard, engineer in charge), have just been concluded. This farm, which covers a surface of about twenty acres, is established approximately on the model of that which has been worked successfully for some time at Brockton, Mass., and as shown in annexed plan, is divided into regular beds, separated by small earth embankments so as to divide the irrigation according to requirements of cultivation and amount of sewage to be distributed.

The sewage water flows by gravitation into a well which communicates with two tanks placed beside each other. These tanks can be used in turns; a movable grate in each of them stops the passage of insoluble matter. From the tank the sewage water

* By the oxidation of the organic matter.

passes into two parallel earthenware pipes, eighteen inches in diameter, which distribute it by means of earthenware gates on any of the beds when it may be required. At the end of the pipes there exists an overflow arrangement which can spread any surplus water on a large surface of ground, to be devoted for the present to rough cultivation.

The soil has been regularly levelled and subdrained by means of farm tiles covered with a foot of coke. These farm tiles are at about five feet below the surface, and end in an open ditch made of brick which in its turn pours its contents into another ditch when they finally flow into the Back River.

The soil of the farm is approximately constituted as follows:—

1. One foot loam.
2. One foot loam and gravel.
3. Two and one-half feet light clay mixed with sand and gravel.
4. One and one-half feet quicksand.

The impermeable bottom is hard pan. This farm will receive at present the sewage and a portion of the surface water of a district containing actually a population of about 2,000.

To begin working it the only thing now required is the completion of the main sewer which is to convey the sewage to it, and which will be finished at latest next spring.

The writer believes he has now covered all the important points of the subject. He has gone through the theory of purification by the soil, and has given an insight into some of its practical applications in the old world, the United States and Canada. He believes, therefore, he is fully justified in concluding with the following statement:

A municipality called upon to pronounce on the choice of a system for external sanitation can only proceed on the path followed by all the cities which have had to solve this grave problem by adopting the same sanitation methods. They have been tested and their use confirmed by long experience, and have given the most complete satisfaction, either in reference to health or for the proper agricultural utilization of the quantities of refuse produced in cities.

But it may be objected that the realization of the exterior sanitation of cities will cause heavy expenditure.

That is true, and yet municipalities should understand that, if they have to pay for bringing in pure water, they must equally, and the obligation is no less strict, pay a second time to remove this same water encumbered with the refuse of the town, and finally pay again to purify it by regular and intermittent irrigation on a soil both suitable and cultivated, so that, whilst harmless to the neighbourhood, it may be of use to cultivation.

What municipalities will spend on internal and external sanitation they will recover in the lengthening of human life and the diminution of the budget of hospitals.

CAN. SOC. C. E.
PLATE V

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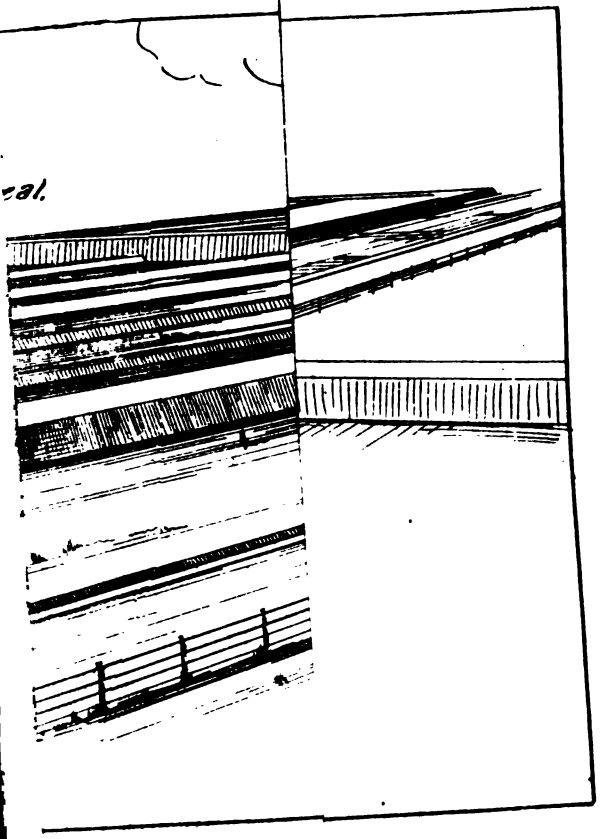

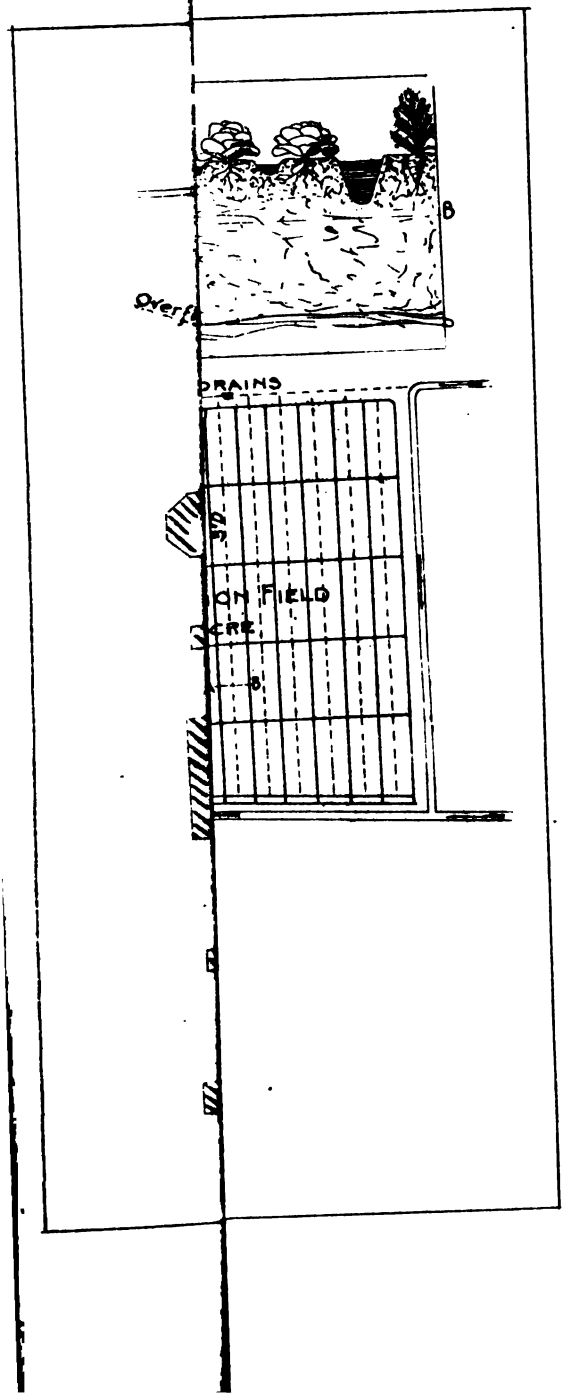


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## DISCUSSION.

Mr. Stuart Howard.

The vexed question of sewage disposal has within the last few years become very serious, the contamination of the rivers and streams, from many of which cities, towns and vilalges derive their drinking water, has been prevented by law, and the question then arises, what to do with the sewage. Many of the older places in England, France, Germany, and even on this continent, have adopted some system of sewage farm, or sewage destruction, either intermittent filtration, flooding, chemical or septic tanks. Especially is it so in the State of Massachussets, every outlying district of Boston adopting one or other of the systems, the whole country for miles round being by the State converted into a system of receiving reservoirs for the supply of water to the city itself and neighbouring towns and villages, so that the emptying of drains into the water courses has been prohibited. Brockton has a model sewage farm, situated close to the main thoroughfare, along which stand the residences. No disagreeable odour is apparent, and the crops grown are most prodigious. The sewage farm lately constructed for the disposal of the sewage from St. Denis Ward of the City of Montreal, has already ten acres prepared, and can be increased in size as the place grows.

This farm was built by Mr. Janin as contractor, and it was mainly through his efforts and advice to the Road Committee that the scheme was adopted, and there is no doubt it will be a success, and the means, the writer trusts, of other places following suit

The question has been asked regarding the water in times of freshlets and heavy storms. The writer may say that St. Denis Ward has been laid out with a separate system, only a sufficient amount of water to flush the sewage being allowed to enter the main drains.

It has also been asked as to the acreage of farm required. This question has been thoroughly discussed in England, and decided that one acre is sufficient for the sewage of from 500 to 600 inhabitants.

Mr. Janin's paper would be made doubly interesting if he would give the approximate cost of preparing an acre of farm, and also the cost of all the different systems, so that anyone dealing with the matter of sewage disposal, would be able to arrive at the most economical, as well as the most advantageous. Many of the questions relating to the action of microbes, etc., will, the writer believes, be thoroughly answered in Mr. Lea's paper, which is the next to be read before the Society.

Thursday, 26th April.

DR. H. T. BOVEY, President, in the Chair.

The following report was received from the scrutineers of the ballot for the election of Members on April 12th:—

"We, the undersigned scrutineers, beg to revise our count of the 12th instant, by adding the name of H. T. Morrison as Associate Member.

J. S. VINDIN.
R. T. GOUGH."

It was moved by Mr. E. A. Rhys-Roberts, seconded by Mr. H. Irwin, and carried: "That the revised report of the scrutineers of the ballot for the election of members be accepted."

It was moved by Mr. W. McLea Walbank, seconded by Mr. W. McNab: "That the Society should express to the Corporation of the city its opinion that the office of Building Inspector should be filled by a competent engineer or architect."

A committee, consisting of Messrs. W. McLea Walbank, W. McNab and C. H. McLeod, was appointed to draft and forward a resolution in connection with the foregoing.

Messrs. E. A. Rhys-Roberts and G. Legrand, having been appointed scrutineers of the ballot for the election of Honorary Member, declared Lord Strathcona and Mount Royal duly elected as Honorary Member.

*Paper No. 150.*

## SEWAGE DISPOSAL.

By R. S. Lea, Ma.E., M.Can.Soc.C.E.

It is a remarkable fact that, while Damascus, Nineveh, Babylon, and many other ancient cities of the old world, were provided with abundant artificial water supplies and extensive systems of sewerage, it is only within the last half-century that the drainage system of London, the greatest of modern cities, has been brought to such a condition of efficiency as to permit of its general use in the removal of household waste.

During the long, dark period of the Middle Ages, which succeeded the dismemberment of the Roman Empire, sanitary provisions were almost entirely neglected, the narrow streets and alleys of the crowded cities were made the receptacles for the waste products of the teeming populations, and bodily filth even came to be regarded as a mark of holiness of mind. The penalties for these ignorant violations of natural laws followed in the form of terrible plagues and epidemics, while the unfortunate misconception of the nature of the offence suggested no more effective remedy than the wearing of sackcloth and ashes.

In London, during the seventeenth and eighteenth centuries, rough drains were constructed for the removal of surface water; but for the household waste or sewage proper the gutters, occasionally supplemented by pits or cesspools, remained the only method of disposal. In later years, as the methods of construction improved, the storm-water drains were sometimes used for sewage as well; while earth closets and various pail systems for direct removal began to be substituted for the vaults and cesspools.

Meanwhile an artificial water supply had been introduced, and in 1847 the drainage system had been improved and extended to such a degree that it was made compulsory to discharge into it all sewage of every description. This was a return to the water carriage system of the ancients, in which the liquid wastes serve as a vehicle for the removal of the solid, with manifest advantages from an economical as well as a sanitary point of view. Yet, through a desire to utilize the manurial qualities of the excremental matter, many places continued for several years to employ some form of Direct Removal, without the admixture of water, for this portion

of the sewage. But the objectionable features unavoidably connected with such methods, together with the increase in the number of public water supplies, soon tended towards the general adoption of the cleanly and efficient water carriage system, which, in America at least, may now be said to be almost universal. The assumption that such is the case will therefore be made in the further discussion of the subject matter of this paper.

Having provided the drains for its collection and transportation to some distant point, the first and most natural method of disposing of sewage in this highly diluted form was to turn it into the nearest body of water; and this was usually done without much regard to its volume. This system, while generally proving quite satisfactory to the community employing it, very soon began to develop into a nuisance for many of its neighbours, and as the construction of drainage systems became more general, it was soon evident that there was a limit to the quantity of sewage which could be disposed of in this way. Not only did many of the rivers soon become unfit for sources of water supply, but some of the smaller ones were practically becoming converted into large open sewers, disgusting in appearance as well as dangerous to health.

The small streams, dense population, and extensive industries of England, rendered her particularly liable to suffer from this condition of things. A Royal Commission, appointed to inquire into the question of stream pollution, made its first report as early as 1870, and in 1876 the Rivers Pollution Bill was passed, making it compulsory to purify sewage by some kind of artificial treatment before discharging into a watercourse. The passage of such an Act naturally led to the appearance of a great number of so-called "processes," patented and otherwise, for the treatment of sewage. It also had the effect of stimulating the investigation by scientific men, into the nature and constitution of the particular substances which gave to sewage its offensive and dangerous character.

This was a period of remarkable progress in the science of biology and organic chemistry. The processes of fermentation and the decomposition of organic matter were shown to be, under some conditions at least, dependent upon the action of living organisms, and not purely chemical, as was previously thought to be the case. Further investigation has proved this to be true, under all conditions and circumstances. The ideas and theories brought forward were soon put to practical test in the construction of a number of working plants, and as the studies and investigations have proceeded the results from these works have been of the greatest assistance in pointing out the direction in which the researches should be pursued.

In France and Germany the subject has been developed along the same lines, but in America the necessity for such action was at first by no means so evident, by reason of the relatively sparse population and the immense size of the rivers and lakes. Nevertheless, in some of the New England States, the conditions began to be approximately similar to those common in Europe, and the problem of preventing stream pollution had already occupied the attention of the legislatures of some of these States, notably Massachusetts, previous to 1887, when the latter State undertook the well-known series of experiments which have since been carried on under the directions of its Board of Health. As a general thing, however, the matter was given little consideration. Occasionally, when a water supply became seriously threatened, the subject was given a certain amount of discussion, but the tendency was, in most cases, to look upon the matter as an indication of the necessity for a new water supply rather than for special treatment of sewage, and to regard the pollution of rivers in this way as rather in the natural order of things.

Within the past few years, however, the increase and spread of sanitary knowledge has brought about a change in these ideas, and since 1890 several effective systems of artificial disposal have been built, and are in operation in different parts of the country. Lately, a period of protracted drought has compelled several cities to resort to the use of polluted river water, in order to reinforce their ordinary supplies, and it has become evident that the normal growth will soon compel many of them to turn to the large rivers for their permanent supply. In view of this certain outcome, and of the fact that the rate of river pollution will be much in excess of the growth of population, the wisdom of permitting the continuance of such pollution is at least questionable. In Connecticut and New Jersey legislative action during the last few months has placed the control of the streams in this respect in the hands of bodies appointed for that special purpose.

Evidence of a similar change of public sentiment in other States marks the present time as the beginning of a general movement in this direction. Besides, the experience of the last twenty-five years, in conjunction with the results of numerous observations and experiments, have shown conclusively that many of the systems hitherto proposed are unscientific in principle, as well as incapable of effecting the required degree of purification in an economical manner. So that we have now arrived at a stage where the matter is simplified by the clearing of the field of a number of elaborate contrivances, expensive as well as inefficient.

The evidence all points to the conclusion, that the nearer the process can be kept to that of Nature the more satisfactory will

be the results. In fact, in the light of present knowledge, it is probably not too much to say that there is really but one feasible and effective method for the treatment of sewage; or rather that those processes which have been shown to be capable of producing the results required in an economical manner, are based upon the same general principles.

## ORGANIC MATTER.

Of the different substances composing sewage, only those of organic origin give rise to the necessity for its purification. Hence, by considering the true nature of organic matter, and the changes it undergoes, an idea may be obtained of the operations actually taking place in any of the various purification processes, as well as the essential features of each method, and the conditions necessary for its proper working. With this object in view the accompanying diagram (A) annexed, has been prepared, as a graphical representation of the main facts relating to the origin and constitution of organic matter, and of the subsequent transformation and changes which sooner or later it must pass through.

Organic matter includes all those combinations of the chemical elements whose formation depends upon the processes of life, and which are therefore derived from the substance of either plants or animals. The various forms which it may assume are set out, in the order in which they occur, around the arc of the circle in the diagram,—the direction in which the changes proceed being shown by the arrows. The vertical line at the top of the diagram divides these processes into a period of life and growth, and one of death and decay. At the bottom of the diagram, below the line marked "complete oxidation," is represented the inorganic material of the earth's crust, which consists almost entirely of inert mineral substances, whose chemical affinities are completely satisfied, and which are therefore without any kind of available energy. The meaning of this will be evident from the following illustration:—

When a weight is raised or a spring compressed, an expenditure of energy is necessary, because work must be done,—in the one case against the force of gravitation, and in the other against the molecular forces which resist change of shape in elastic bodies. The compressed spring and the lifted weight are now said to possess potential energy, because if the one is allowed to assume its original form, or the other to descend to the earth, the energy which has been expended upon them will, in some form or other, be set free again. And it must be further observed that there always exists the tendency towards the liberation of energy stored in this way,

whenever the opportunity is offered, which renders the arrangement one of *instability.*

Exactly the same condition of things exists when the chemical affinities (attractive forces) of some of the atoms of a compound for oxygen, say, are unsatisfied. So long as this combination is prevented from taking place, so long may the compound be said to be in a condition of instability, and to possess potential energy, which will become available whenever the oxidation is allowed to take place. This energy is of the same nature and is measured by the same units as that given out by the descending weight or the expanding spring.

Now, the maintenance of life and growth, the power of motion and capacity for doing work, possessed by animals, requires the supply of a corresponding quantity of energy. This we know they obtain from their food, which is ultimately derived from the substance of plants. Plants draw the nourishment necessary for their life and growth from the earth and air, in the shape of materials capable of no further oxidation, and hence entirely devoid of energy. Nevertheless, paradoxical as it may appear, out of such simple, inert materials, the green plants can build up substances, complex in structure, and possessing the rich store of potential energy which fits them for animal food. They are enabled to do this by virtue of a mysterious substance called chlorophyll, which gives them their green colour, and which has the wonderful power of absorbing from sunlight the energy necessary for the purpose.

On the left of the diagram, just above the line of "complete oxidation," is a representation of Plant Life. The space enclosing the words "chlorophyll + sunlight" corresponds to the leaves and other parts of the plant, where this process of food manufacture is carried on. From the earth by way of the roots comes water ($H_2O$) nitrates ($KNO_3$ etc) and other salts in solution; from the air, by absorption through the leaves, carbonic acid gas or carbon dioxide ($CO_2$), and in the case of some plants pure nitrogen. These stable but inert compounds are here broken apart, a portion of the oxygen removed and returned to the air, and the remaining elements built up into *less stable* but *more complex* substances, such as starch, cellulose, sugars, fats, etc. (see diagram). Thus the energy abstracted from the light waves which is expended in this process of deoxidation, is stored up in these substances, which thereby become capable of furnishing food for animals. Animals, by means of their organs of digestion and assimilation, construct from these materials and incorporate in their tissues, compounds still more complex, but of greater instability (see diagram). By oxidation in the lungs or other breathing apparatus, they obtain from them the amount of energy necessary to maintain bodily heat and motion, in just the

same way as energy is supplied to a heat engine by the combustion of fuel. The animal body not being a perfect machine, does not utilize all the energy in its food, but rejects a large proportion of it in the form of excretions, which share the general fate of all dead organic matter represented on the right side of the diagram.

In the manner outlined above, and in a fashion more or less direct is continually being produced "Living Organic Matter," represented on the diagram to the left of the vertical line. And in due course it will join the "Dead Organic Matter" (shown on the other side of the line) either in the form of excretions of living plants and animals, or their bodies after death; and were it not for the processes of disintegration and decay which are everywhere in operation, these constant additions would soon accumulate to such an extent as to check and ultimately stop all life by overwhelming it with its own waste, and by exhausting the materials necessary for its existence. It is this second or *destructive* phase, shown on the right half of the diagram, which is of most interest in this connection, because, as we shall see, it represents the changes which take place in the process of purifying sewage.

## SEWAGE.

We are now in a position to return to the consideration of what sewage actually is.

As it flows in the mains of a modern drainage system it is composed of the discharges from water closets, the liquid waste of kitchens and laundries, the drainings of butcher shops, slaughter houses, markets, stables and manufactories; and, where surface water is admitted, the refuse from streets, backyards, etc. These substances are mixed with a varying quantity of water, which is usually at least as great as the ordinary water supply; so that in spite of the great variety of waste materials which it contains, it is, when fresh, a comparatively innocent looking liquid, cloudy or milky in appearance, but without any particularly disagreeable odour. It is, in fact, dirty water, which contains, partly in solution, and partly in suspension, a certain quantity of organic matter derived immediately from many different sources, but ultimately formed in the manner already described.

The problem of sewage disposal is then the problem of so treating this organic matter, as to destroy it, or render it incapable of offence.

In the diagram "freshly dead organic matter" is shown just to the right of the vertical centre line. As we have seen, it consists almost entirely of the four elements, carbon, hydrogen, oxygen and nitrogen, combined in a great variety of ways. As we have also

seen, it is this complicated arrangement of the atoms, and the relatively small proportion of oxygen, which constitutes the difference, from a chemical point of view, between organic matter and the simple inorganic substances from which it was built up.

The following table (1) by Dibdin shows the actual character of six typical organic substances:—

TABLE I.

| Substance. | Percentage of | | | |
|---|---|---|---|---|
| | Nitrogen. | Hydrogen. | Oxygen. | Carbon. |
| Gelatine..... | 18.3 | 6.6 | 25.1 | 50 1 |
| Chondrine. ..... | 14.4 | 7.1 | 29.4 | 49.1 |
| Albumen..... | 16.0 | 7.1 | 22.0 | 53.0 |
| Cellulose (woody fibre) | .... | 6.2 | 49 4 | 44.4 |
| Starch..... ..... | .... | 6.2 | 49.4 | 44.4 |
| Fat, stearic acid... | .... | 12.7 | 11.3 | 76.0 |

Compounds, like the first three, containing nitrogen, are characteristic of organic matter of animal rather than of plant origin, and are capable of more offensive decomposition. They all contain oxygen, but in relatively small proportions, as will be seen by referring to the chemical formulae of the inorganic substances from which they are derived. The additional amount, which is required to completely oxidize the carbon, hydrogen and nitrogen, is given in table II. which follows (also by Dibdin).

TABLE II.

Pounds of oxygen required to oxidize every 100 lbs. of substance:—

| Substance. | Oxygen required. | | | | Oxygen already present | Difference or additional Oxygen, required for complete oxidation of substance. |
|---|---|---|---|---|---|---|
| | By the Nitrogen | By the Hydrogen | By the Carbon | Total Required | | |
| Gelatine.... ..... | 52.3 | 52.8 | 133.3 | 238.4 | 25.1 | 213.3 |
| Chondrine. ..... | 41.1 | 56.8 | 131.0 | 228.9 | 29.4 | 199.5 |
| Albumen.... .... | 45.7 | 56.8 | 141.4 | 243.9 | 22.0 | 221.9 |
| Cellulose.... .... | .... | 49.6 | 118.4 | 168.0 | 49.4 | 118.6 |
| Starch..... | .... | 49.6 | 118.4 | 168.0 | 49.4 | 118.6 |
| Fat, stearic acid.. | .... | 101.6 | 202.5 | 304.1 | 11.3 | 292.8 |

The small proportion of mineral matters present in these substances is of no importance in this connection, and may be disregarded.

This table shows that the proportion of oxygen in fat, for instance, is so small that nearly three times its own weight of oxygen is required for its complete oxidation. Considering the fact that five cubic feet of ordinary air contains about one cubic foot of oxygen, the above table will give some idea of the quantity of air necessary in some of the purification processes to be presently described.

The complex molecular structure of these organic compounds produced in the processes of deoxidation, has been obtained at the expense of stability, and this implies the existence of a continual tendency towards a re-oxidation of the elements whenever the combinations in which they may be held are broken up. How to produce this in the most direct and economical manner is the object to be aimed at in any system of sewage purification.

If the temperature of an organic substance is raised to what is termed its *kindling point*; in other words, if it is set on fire, the oxygen of the atmosphere will combine with it, and in this manner its oxidation will be accomplished,—the liberated energy manifesting itself in the form of heat. If certain chemicals are brought in contact with it, such as the permanganates, oxygen is set free in the nascent condition, and will combine with it, and so bring about its complete oxidation. Such violent methods are, however, impossible or difficult to accomplish on any considerable scale, as well as uneconomical. Though, in a sense, accomplishing the same object, they are obviously entirely different in method from the mild and equable processes which nature has provided for the removal of the vast quantities of animal and vegetable wastes which are produced from year to year.

## BACTERIA.

These natural forces, which can bring about the oxidation of organic matter even more effectively than those of heat or chemical energy, exist in the bodies of the smallest and humblest of living organisms. These are the bacteria, or microbes, or germs, as they are variously called, which exist in the surface waters, in the air, in the upper layers of the soil; in fact, wherever they are needed. They are of the very simplest structure, consisting of but a single cell, and are of almost inconceivable minuteness. A faint idea of this may be obtained from the fact that a sphere 1-25 of an inch in diameter could contain 500 millions of the larger ones, and it would take more than a thousand of them, placed in a row, to reach across the diameter. Yet, they have been studied and classified into hundreds of different species, whose habits of life, characteristic products, form, etc., are now well known. While individually insignificant, it is only necessary to

consider the enormous quantity of waste material which they are constantly disposing of in order to see how important are the results of their aggregate action.

In shape they are spherical (the *cocci*), rod-shaped (the *bacilli*), spiral-shaped (the *spirilla*). That they may exist in a flourishing and active condition, certain attendant circumstances are necessary, such as presence or absence of air or light, a certain degree of warmth, moisture, etc. As we shall see, they constitute to a great extent the "forces of nature," which the engineer engaged in the problem of sewage disposal must "direct to the use and convenience of man." Other facts relating to them which are of importance in this connection, are as follows:—They are possessed of enormous vital activity, and in the presence of dead organic matter and other favorable conditions, are capable of multiplying with almost incredible rapidity, so that their action is to a certain extent, automatically adjusted to the amount of work they have to perform. It was at first doubtful whether they should be classified as plants or animals; but it is now definitely decided that they belong to the class of colourless plants. They reproduce by fission,—a simple process by which a single bacterium divides itself in the middle, thus producing two. An exceedingly small proportion of them, which includes the pathogenic or disease-producing germs, are *parasitic*,—that is, they exist upon or in the body of a living host. But the great majority are *saprophytic*, which means that they live upon dead and decaying organic matter. A few species are able to exist either as saprophytes or parasites, and are called *facultative*.

Another classification depends upon their ability to live in the presence or absence of free oxygen, e. g.:—

1. *Aerobic*, requiring the presence of oxygen.
2. *Anaerobic*, unable to exist in the presence of this gas.

Facultative anaerobes, which occur in sewage in large numbers, can live either with or without oxygen.

By their methods of obtaining their food, or by the products of their life and action, the saprophytic bacteria—aerobic and anaerobic—are enabled to break up the complex molecules of organic compounds into simple ones, and ultimately bring about their complete oxidation. The process is a gradual one, and is accomplished in successive steps by different groups of bacteria, many of which can act upon the organic matter only when it has been transformed by passing through certain preparatory processes, which have been effected by other groups. It is by reason of this consequent division of the work, together with the fact that the chemical products of one species are often inimical to the life of another, or even to its own when greatly accumulated, that it is possible to modify and largely intensify their action by a certain degree of artificial regulation and control.

## DECOMPOSITION.

The process may be followed with the aid of the diagram, but it must be remembered that it is a very intricate one; and that, therefore, it is possible to show only the most important of the changes which take place, and the order in which they occur.

If the waste material (freshly dead organic matter) is exposed to the atmosphere under such circumstances as will permit all parts of it to have free access to the air, it is immediately attacked by numerous members of the aerobic group of bacteria, which, if other conditions are favorable, thrive and multiply with great rapidity. Besides obtaining the means of supporting their own existence, they cause, by their action in doing so, a breaking up of the chemical combinations of the organic substances, whereby the oxygen of the atmosphere is enabled to combine with the carbon of the compound, producing carbonic acid gas. The oxidation of carbon leaves the nitrogen and hydrogen to unite in the form of ammonia, or of various ammoniacal salts.

This process, the intermediate steps of which are very complicated, and imperfectly understood, is indicated in the diagram by the outer path connecting "freshly dead organic matter" with "ammonia, etc." If the supply of oxygen is not interrupted, a further oxidation of this "ammonia" will take place, the hydrogen to water, and the nitrogen to nitrites and nitrates. This is brought about by means of other species of aerobic bacteria belonging to what is called the nitrifying group. Some of these (the nitrous organisms) can thus change ammonia ($NH_3$), into nitrous acid ($HNO_2$), while others (the nitric organisms) complete the oxidation to nitric acid ($HNO_3$). These acids combine with any bases present, such as soda, potash, lime, etc., to form nitrites and nitrates. This oxidation of the nitrogen of ammonia is known as "nitrification," and is thus the work of certain specific organisms. It has been studied very carefully in connection with the experiments on intermittent filtration of sewage, made by the Mass. Board of Health, the results of which are very fully set forth in its report. Its action, shown by the production of nitrates, appears to proceed to a certain extent coincidentally with the oxidation of the carbon, and is very susceptible to temperature conditions, the supply of oxygen, overworking of the filters, and other circumstances. The nitrates, which, with the water and carbonic acid, represent the final products of this whole process of decomposition, are purely mineral substances, completely oxidized and entirely without energy. These operations, therefore, represent the complete destruction of organic matter, by its successive conversion into substances of less and less complexity of structure, but of greater and greater stability, until it is finally re-

solved into the water, carbon, dioxide, and nitrates out of which it was originally built up, and in which form it is again available for use in the structures of other plants and animals.

These operations, which are truly as old as the hills, have always been familiar to us, though until very recently entirely misunderstood. The changes taking place were thought to be purely chemical, and brought about by mere exposure to the air, or, as in the case of the rapid disappearance of manures in cultivated lands, by the action of the growing plants, and of the soil itself. This view is now known to be entirely incorrect, and it has been repeatedly shown that, however abundant the air supply, no decomposition can take place without the presence of bacteria. On the other hand, as we shall presently see, certain of these micro-organisms are able to set up a process of decomposition in organic matter, and even produce a certain amount of oxidation, under circumstances where air is carefully excluded.

## PUTREFACTION.

It is decomposition under such conditions as these which we shall next consider. The breaking up of the organic compounds is now brought about by various species of the anaerobic bacteria, which find in the exclusion of air and light just the conditions necessary for their development. Their action does not proceed with the same outward simplicity as that of the aerobes, but is accompanied with the production of offensive odours, noxious gases, and other disagreeable manifestations characteristic of what is termed putrefaction.

It is this kind of decomposition that goes on in organic deposits at the bottom of stagnant pools, in cesspools and the like,—and wherever nitrogenous organic matter is prevented from having access to air. Even where the decomposition is to outward appearance entirely aerobic, the preliminary disintegration which enables the air to get free access, must often be of this nature.

The chemical changes are entirely different from those taking place in aerobic decomposition. Certain species of the anaerobes have the power in the presence of organic matter of de-nitrifying nitrites and nitrates,—that is, deoxidizing them. When the circumstances are favorable to such action the nitrogen is set free as nitrogen gas, while the oxygen attacks the organic matter, bringing about the formation of carbon dioxide and other gases. The chemical changes are at first, however, especially in the case of sewage, of the nature of what the chemists term *hydrolysis*, or the breaking up of organic compounds by the action of water. This action often results in the liquefaction of refractory and otherwise

insoluble substances, like fibrin or cellulose, which process is accompanied by the formation of many gaseous compounds of carbon, hydrogen and nitrogen. These, and many other chemical changes, which take place according to the attendant circumstances, are described in detail by Dr. Samuel Rideal in the Journal of the Society of arts.* Referring to the anaerobic decomposition of the organic matter in sewage, he says:—"Very similar changes, mainly hydrolytic, are accomplished by the large class of organic substances called 'enzymes,' which, though not living, are products of animal and vegetable life. . . . Their importance to us is shown by the fact that a large number of them are the products of the bacteria or other fungi, and are powerful agents in their resolving action. By their means a bacillus is not only able to act in its immediate neighbourhood, but also at a considerable distance, through the soluble ferments it forms and disengages."

As indicating the complex nature of the process his chemical classification of the "fermentations which occur in the first or hydrolytic part," is quoted as follows:—

"1. The solution and decomposition of albuminous bodies.

"2. The fermentation of urea.

"3. The fermentation of amido-compounds formed from the albuminous bodies.

"4. The formation of organic acids and the fermentation of their salts.

"5. Cellulose or methane ($CH_4$) fermentation.

"6. The hydrolysis of carbohydrates (sugars, fats, etc.).

"7. The formation of small quantities of sulphur compounds, like $H_2S$, mercaptan, etc. This from the odour of the products often attracts the most attention."

This process is indicated in the diagram by the first part of the inner path connecting "freshly dead organic matter" with "ammonia, etc." It takes place very rapidly, as compared with purely aerobic processes, and results in a very considerable reduction in the amount of organic matter, in the formation of volatile gases, etc. Though the changes which have taken place are in the right direction, the process of decomposition is as yet very incomplete. Many of the resulting products are highly offensive in character, but the solid portions have been largely liquefied, and otherwise altered so as to bring them into a condition which renders them readily oxidizable.

If, now, in the natural course of events, or by artificial means, air is brought in contact with them, the volatile gases will be dissipated, and gradually the aerobic bacteria will take up the work and carry it on, as shown by the diagram, to the points where nitrification

*Vol. XLVI, p. 81, Dec., 1897, and Vol. XLVII, p. 683, Jan., 1899 (Cantor Lecture)

begins; which, with the final oxidation of the nitrogenous and carbonaceous residues of the previous processes, proceeds in much the same way as that already described in connection with aerobic decomposition.

The two processes outlined above, though beginning under opposite conditions, and proceeding in an entirely different manner, achieve in the end the same result, viz., the destruction of organic matter by its conversion into stable, gaseous and mineral substances. They are nature's methods, and have been in operation over the surface of the earth since the first appearance of life upon it.

Under conditions of abundant aeration they are without any disagreeable or other noticeable features. Where the air has no access, putrefaction takes place, and when complete and efficient drainage systems were first constructed, it was this tendency of the sewage to putrefy when discharged broadcast into relatively small streams which created the demand for its purification. Ignorance of the true nature of the processes involved led to many attempts to accomplish this by purely artificial means, but experience has shown, that these efforts to supplant nature on such a large scale have always resulted in practical failure.

Since that time the investigations of the biologists have shown the reasons why they have failed, and at the same time have led to the discovery and employment of methods which are proving successful, because, by providing the most favorable conditions and removing or holding in check those which operate adversely, they assist nature in every way; the result being that purification takes place at many times the rate possible when not thus controlled. To this extent only are such methods artificial.

## CHEMICAL ANALYSIS.

Before proceeding with a description of these methods, we must refer to the subject of the chemical analysis of sewage; because it is by means of it that it is possible to follow the progress of purification, to compare the working of different processes, to determine their efficiency by testing the effluents, and to discover, when these are not satisfactory, at what stage or in what way the process is going wrong.

The principal determinations in a chemical analysis are given under the following heads:—

1. Appearance.
2. Total Solids or Residue on Evaporation.
3. Loss on Ignition.
4. Chlorine.
5. Albuminoid Ammonia.
6. Free Ammonia.

7. Nitrogen as Nitrites.
8. Nitrogen as Nitrates.
9. Oxygen absorbed, or consumed.

Under "Appearance" are noted the turbidity, colour, etc., which are expressed in terms of some standard, depending upon the method of observation.

The amount of the total solids is obtained by evaporating a portion of the sewage to dryness, and weighing the residue. The latter will be partly organic and partly inorganic. By carefully heating it to a low red heat a portion of it will be driven off in the form of gases. The remainder is weighed, and is called the "fixed residue," while the part which passes off is called the "loss on ignition." The amounts indicate roughly the relative proportion of the inorganic and organic matters respectively. Sometimes the sewage is first filtered, and the above determinations made separately for the "dissolved" and "suspended" matters.

Common salt is one of the characteristics of domestic sewage, and it is very largely from it that the "Chlorine" in the analysis is derived. It is thus a measure of the strength of the sewage, and is useful in determining the identity in this respect of a sample of a putrified effluent with the original sewage.

The most important information, however, which a chemical analysis furnishes, with regard to the relative purity of sewage, and effluent, and of the manner and rate at which the process is taking place, is that given under the next five headings,—"Albuminoid Ammonia," "Free Ammonia," "Nitrogen as Nitrites and Nitrates," and "Oxygen Absorbed."

We recall that organic matter is composed almost entirely of carbon, hydrogen, oxygen and nitrogen, combined in various ways, and produced by the agency of green plants by processes of *deoxidation*; and that, therefore, the reverse process of decomposition is one of *oxidation*, requiring a supply of oxygen from external sources, taking place in a variety of ways, but always depending upon the activities of the bacteria. Also that the result of the first step, is the oxidation of carbon, and the formation of various compounds of nitrogen, such as ammonia, ammoniacal salts, etc.; and that then other bacteria bring about the oxidation of the nitrogen also, forming first nitrites, and lastly nitrates, which are mineral substances, and capable of no further oxidation.

If water or sewage containing organic matter in process of decay, is boiled, the ammonia which has been produced, either in the free form, or as salts, will be driven off, and may be obtained by distillation. The amount (which is given in the analyses as "Free Ammonia") is thus a measure of the extent to which the first step in the decomposition has taken place. The nitrogen of *fresh* organic matter may also be obtained in the form of ammonia if it is

boiled in an alkaline solution of permanganate of potash. The quantity of ammonia obtained in this way, after the "Free Ammonia" has been driven off, is given in the analysis as "Albuminoid Ammonia." It is thus a measure of the amount of organic matter present, which has not yet begun to decay, and it is thus shown in the diagram. A decrease in the amount of albuminoid ammonia with a corresponding increase in that of free ammonia would indicate progress in the first stages of purification.

The amount of nitrogen present in the form of "Nitrites" is determined, and, being so given in the analysis, represents the beginning of the oxidation of the nitrogen, and hence further progress in decomposition, while the "Nitrates" represent the completion of the final step in the process of purification.

There is another determination which is useful in ascertaining the degree of purification, viz., the "Oxygen absorbed." This is obtained by treating the sewage with an acid solution of permanganate instead of an alkaline, as in obtaining the amount of albuminoid ammonia. The oxygen of the permanganate attacks the organic matter, and combines with the carbon. The total quantity of oxygen so absorbed in a given number of hours, is measured, and will be greater or less in proportion to the amount of unoxidised organic matter present. Since in this process the oxygen given up does not combine with the nitrogen, the "Oxygen absorbed" will correspond to the carbonaceous organic substances rather than to the nitrogenous, which are represented by the albuminoid ammonia. The refuse from manufactories often contains mineral substances, which absorb oxygen very readily. In such cases the "Oxygen absorbed" would be no indication of the quantity of organic matter. The determination is of greatest use in comparing the quality of an effluent with the raw sewage. The percentage reduction in the "Oxygen absorbed" will be then a measure of the degree of purification, which latter is usually thus stated, in terms of either the "Albuminoid Ammonia," or the "Oxygen absorbed," or both.

Under ordinary circumstances, pure water contains a certain amount of atmospheric oxygen in solution. If decaying organic matter is mixed with it, oxidation would take place, provided the bacteria were present, at the expense of the dissolved oxygen, and hence the diminution in the amount and the rate would furnish some information as to the condition of the organic matter, and the changes taking place in it. In watching the progress of purification, therefore, the determination of the amount of "Dissolved oxygen" is often of great importance.

In making the Manchester experiments, which will be referred to further on, an "Incubator Test" was employed to test the quality

of the sewage effluents, canal water, etc. It is thus described in the experts' report:—

"A very valuable indication of the degree of impurity of any sewage or effluent is afforded by the so-called incubator test. To carry out this test a determination is first made of the oxygen absorbed from potassium permanganate by the sample in three minutes. A bottle is then completely filled with the sample and closed, and placed in the incubator at 80° Fahr. for six or seven days. The three minutes' absorption is then again determined. If any putrefaction has taken place, the oxygen absorbed in three minutes will exhibit a decided increase in amount, owing to the more ready oxidizability of the products of putrefaction, such as sulphuretted hydrogen, etc. On the other hand, if the sample keeps sweet, the three minutes' absorption remains practically unchanged after incubation, or there will be a slight decrease, owing to slight oxidation of the impurities which has taken place during the period of incubation at the expense of the nitrate or dissolved air present in the sample."

In the Diagram (A) the determinations "Albuminoid Ammonia," "Free Ammonia," "Nitrites" and "Nitrates" have been placed so as to indicate as nearly as possible the various stages in the process to which they correspond. The "Oxygen absorbed" is placed where it is, because the fresher the organic matter the higher will be the amount.

The above determinations furnish the means by which the action of sewage purification processes are tested and compared, and are therefore of the greatest importance; but unless care and judgment are exercised in collecting representative samples for analysis the results may be without value, or entirely misleading

We are now in a position to refer briefly to the various methods which have been devised since the necessity for sewage treatment first became evident, to accomplish that end, and to describe the principal features of those which, being based on true principles, have proved their practicability and efficiency.

## ARTIFICIAL DISPOSAL.

After the report, in 1870, of the Royal Commission, mentioned above, a very great number of methods were invented, most of which were based on the idea that the process of decomposition was purely chemical, and could therefore be effectively promoted and carried through by methods entirely artificial. It was also thought that substances incidentally produced would have high fertilizing qualities, and that, by utilizing them as manures the purification of sewage might be carried on without cost, or even at a profit.

The extreme variety in the nature and condition of the organic matter in sewage, and the large proportion of water, rendered any process which aimed at its oxidation solely by chemical means while thus diluted, entirely impracticable. The same may be said of attempts in the direction of disinfection or sterilization, with the object of arresting decay, or rendering the products innocuous. Chlorine in various combinations was used for this purpose, principally in the form of chloride of lime or bleaching powder. The so-called electrolytic processes are of this nature. By electrolyzing a mixture of sewage and sea-water, or a solution of common salt, a result is produced similar to that of the action of bleaching powder. The oxidation produced, which is limited, is due to the nascent oxygen set free by the action of the chlorine in decomposing water. The Webster process of England, and the Hermite of France, are among the most important of these processes.

The effluents from such processes are very imperfectly purified, and the germicidal action is a direct hindrance to the work of the bacteria, which must ultimately complete the decomposition.

## CHEMICAL PRECIPITATION.

The chief difficulty in any method of treating sewage is with the "sludge," or the portion composed of the solid matter, and the larger and heavier of the suspended particles, which tend to separate from the rest by subsidence. Those chemical processes which have survived the experimental stage had for their object the production of a clear effluent by the removal of the sludge, and as nearly as possible all of the suspended matter. This was effected by mixing certain chemicals with the sewage, and running the mixture into large tanks, where it remained long enough to permit of the precipitation of the suspended matter. A great number of chemicals have been employed for this purpose, which accounts for the hundreds of patents granted at the time these methods were first being tried, but experience has shown that lime and sulphate of aluminum or sulphate of iron are the most satisfactory from the point of view of economy and efficiency. In the settling tanks the action of these chemicals upon certain substances always present in sewage, and upon each other, results in the formation of insoluble flocculent precipitates, which, as they fall, entangle and carry down with them almost the whole of the suspended matter. There is very little reduction, however, in the quantity of dissolved substances, so that the effluent liquid, which is drawn off from above, while clear, still contains nearly half the organic matter of the raw sewage, which is subject to putrefaction, and which must still pass through the various processes of decomposition before

it can be said to have been purified. Of course, such an effluent is much less liable to cause a nuisance when discharged into a stream than the raw sewage, and the quantity of organic matter being reduced by a half, it may often be discharged with safety into a much smaller stream. But the large quantity of sludge from the bottom of the tank has yet to be disposed of, and it is here that the great objection to chemical precipitation exists. It also is subject to putrefaction, though the process has been hindered by the destruction of a great number of the bacteria. But their destruction is not complete, and they soon reassert themselves, so that the sludge soon becomes very foul. It still contains a large proportion of water, and is thus very difficult to deal with. Sometimes this water is partially removed by filter presses, or by allowing it to drain away and evaporate in shallow sludge basins, or on gravel filter beds. The remainder is sometimes used as manure, either as it exists, or after being mixed with other material to improve its quality. It has been used with clay to form cement and bricks. Often the sludge is taken directly from the tanks and spread over large areas of loose soil into which, when dry, it is ploughed. The manurial value is low, since the fertilizing elements in sewage are largely in solution. In London and Manchester it is pumped into tank steamers and carried out to sea. In adition to the difficulties in conducting the process, such as properly proportioning the quantity of chemicals to the varying strength and character of the sewage, the disposal of the sludge is always an expensive matter, not to speak of its disagreeable features. Besides this, in many cases the effluent does not satisfy the local requirements as to purity, and has to be further treated by irrigation or some other process in which the bacteria—the true purifiers—are enabled to act. Hence, in the light of what is now known of the true processes of decomposition, it is evident that the treatment of sewage by chemical precipitation is not a process of purification at all, but merely a separation of it into two portions, both of which are still to be disposed of.

Nevertheless, a great number of such plants have been built in Europe, chiefly in England; and a few in America. In nearly every case they have proved unsatisfactory, and have been abandoned, or are being abandoned in favour of processes which work with nature instead of against her. An excellent example of such a plant, both as to construction and operation, is that of the Worcester, Mass., sewerage system. The reports of the department contain much detailed information as to efficiency, cost, management, etc.

## SEWAGE FARMING.

Another method that was early employed with the object of disposing of sewage, and at the same time obtaining a return from its manurial qualities, is known as Broad Irrigation, or Sewage Farming. This is to a certain extent a natural method, and consists in applying the sewage to large areas of cultivated land, in such small quantities that the supply of air in the pores of the surface soil is always sufficient for the conversion by the bacteria of the organic matter to inorganic, in which form it is available for plant food. The action is thus chiefly aerobic. Unless the subsoil is very porous the surplus water must be removed by under-drainage.

With favourable soil, and careful and intelligent management, the method is satisfactory. Some of the difficulties connected with it are the tendency of the particles of sludge to choke up the surface; the variable needs of the crops for water depending on the rainfall, the season of the year, etc., the effects of frost and snow; and the tendency to become at times a nuisance to people living in the neighbourhood, even at a considerable distance. It requires large areas of land, often very expensive or impossible to obtain; but when well and carefully managed, it is a decided success as a purifying process.

It was at first undertaken with the idea that plant life of itself was capable of decomposing the organic compounds contained in sewage. Later, however, it was discovered that they had nothing to do with it, and could make no use of these materials until the decomposition had been first brought about by the bacteria. Still it was thought that such soil, teeming as it is with bacterial life, would be the best material to employ. But it was soon found out that quartz sand or any such porous material would acquire the same purifying power simply by dosing it with the sewage for a period of two or three weeks. Not only that, but the amount which it could purify in a given time was found to be very much greater.

## INTERMITTENT FILTRATION.

The sand itself has no purifying power, but the sewage comes laden with bacteria which find on the surface of the sand grains an extensive area upon which to establish themselves, the air in the spaces between the grains supplying them with the necessary oxygen. If the application of the sewage is continuous, this air is soon used up, and then purification becomes very imperfect, putrefaction ensues, and the sand becomes foul and choked up. But if the sewage is applied intermittently, it draws in after it, as it sinks, a new supply of air, which becomes available for the next dose,

and so on. Worked in this way, the arrangement is called an Intermittent Filter. It is not a filter at all, of course, since when properly working the action is entirely biological; that is, carried on by living organisms. It is shown on the right side of diagram A, beginning with decomposition by aerobic bacteria, and ending with the completion of nitrification. The degree of purification under favorable circumstances cannot be surpassed, the effluents from the under drains of the "filter beds" being often chemically purer than many domestic water supplies. In practice the area is divided up into beds, upon any of which the sewage may be turned at will, from the distributing pipes or channels, which are placed in the embankments separating the beds. Under-drains at a depth of four or five feet collect the purified effluent. In some exceptionally favourable situations, where the subsoil is coarse sand and gravel, and near a valley or depression, the natural underdrainage is sufficient.

With good material as much as 100,000 gallons per acre per day can be purified. If much more than this is applied, even under the best conditions, the capacity of the bacteria will be overtaxed. and an imperfectly purified effluent will be the result. In most cases, however, the amount per acre is very much less than this, on account of the inferior nature of the material usually available, and by the choking of the surface by sludge.

The heaviest and coarsest portion of the suspended matter is usually removed by screening and simple subsidence for a short time in small tanks. The rest can usually be disposed of by the beds, if they are not overworked, and if the surface is dug over and loosened up when signs of clogging appear. The surface should be ridged and furrowed before cold weather, so as to allow the ice which forms to rest upon the ridges, roofing over the hollows, and preventing the sewage from freezing. Crops of vegetables are often grown upon the beds successfully. This is not a part of the purification process, and is therefore of secondary importance.

Intermittent filtration is therefore in accordance with the methods of nature, and the comparatively high rate at which these filter beds may be worked, is obtained, 1st, by selecting material the most favourable for the action of the bacteria; 2nd, by applying the sewage so as to always maintain a sufficient air supply; 3rd by removing the coarse solids, etc., from the sewage before it is turned on the beds; 4th, by careful personal attention to the working of the plant and prompt correction of any wrong tendency as soon as it appears. In principle, however, it is an aerobic process, and therefore does not provide adequately for the disposal of the sludge. This is usually, when it is settled out, run on to sludge beds, where,

after drying, it is composted or otherwise disposed of. The amount requiring such treatment is, of course, much less than that from a chemical precipitation plant. Nevertheless, it may become a nuisance to the neighbourhood, and will necessarily accumulate during cold winter weather; so that the lack of provision for it constitutes a defect in the system, though a comparatively slight one, when its superiority to the majority of other systems in use is considered. The advisability of employing it depends largely upon the existence of suitable material, since, on account of the great area necessary, it would be a very expensive matter to construct such beds of material transported from a distance.

In the State of Massachusetts, for instance, this method is in favour because suitable material is available in nearly every locality. The beds are prepared by stripping off the surface soil and levelling up the beds of sand and gravel thus exposed. On account of the prevalence of these favourable conditions, the experimental work of the Board of Health experts has been largely with this method of purifying sewage. In England, on the other hand, clayey soils predominate, and hence neither intermittent filtration nor sewage farming have been very successful. Land treatment of the effluents from chemical precipitation plants is not uncommon, and it is required at the present time by the Local Government Board for those of the Bacterial Systems to be referred to presently. Hence, in England, particularly during the last ten years, many investigations and experiments have been made upon the purifying action of other material than sand, such as coke breeze, cinders, ashes, burnt clay, etc. The object was to discover some method by which these materials could be utilized to purify the sewage at such a rate as would keep the first cost within reasonable limits. Such experiments have been carried on by the Main Drainage Committee of the London County Council at Barking and Crossness, and by the authorities of Sutton, Exeter, Leeds, Sheffield, Manchester, Birmingham, and other places. These investigations, which have been watched with the greatest interest by the many other places with similar problems to solve, have been remarkably successful, and have resulted in the discovery of more than one system in which the whole of the sewage,—not the finer and liquid portions only, but the sludge as well—may be purified from first to last by natural processes and at a comparatively low cost.

## NEW BACTERIAL METHODS.

The distinguishing feature in these processes is the recognition of the work of the anaerobic bacteria, whose development and activity depends upon the exclusion of air and light. In other bacterial

processes constant and abundant aeration is provided to as great an extent as possible. The aerobes, which develop under these encouraging conditions are very effective in purifying the organic matter in solution, but are less successful with even the finer matter in suspension, and fail altogether to deal with the larger solids and such substances as paper, rags, strings, etc., which must be separately disposed of. By the new methods, however, it is shown that this separation is not necessary if the whole of the sewage is first worked over by anaerobic bacteria. The result of their action is to bring the organic matters, solids as well as liquids, into such a condition that they can be readily disposed of by the subsequent action of the aerobes, and at a much greater rate. In so doing they furnish the best solution so far known of the difficult problem of the disposal of sludge.

When sewage is first discharged from the houses, streets, etc., into the sewers, it contains a considerable amount of dissolved oxygen. During the time the sewage is flowing through the mains toward the outlet, the aerobic bacteria, in the presence of the oxygen, begin their work of decomposition, which, as we have seen, results in oxidation of carbon and the formation of ammoniacal substances. This action will proceed until the dissolved oxygen is all used up, which it soon will be, but in the meantime very little putrefying action will have taken place, and that is the reason why the sewage in the pipes of a well-designed system is comparatively free from offensive odours. Chemical analysis would show during this period a slight decrease in the albuminoid ammonia, with an increase in the free ammonia, and perhaps in the nitrates. When the oxygen is exhausted, anaerobic action begins, resulting, as we have seen, in the breaking down and liquefaction of a portion of the solids, and the production of various gases, some of them very foul-smelling. This action will sometimes have begun before the sewage reaches the outfall, the degree of which has taken place depending upon the length and inclination of the mains.

The fact of the disappearance of the solids in sewage under circumstances in which aeration is impossible, was known many years ago. About twenty years ago there appeared in the French *Cosmos les Mondes* references to what was called the "Mouras Automatic Scavenger." This was described as consisting of a closed vault with a water seal, in which all the excrementitious matters were rapidly transformed into a homogeneous fluid; and it was stated that in time kitchen refuse, onion peelings, paper, etc., also dissolved and disappeared. It was said to be automatic in action, and to have been in operation for twenty years.

In 1870 Frankland experimented on London sewage by causing it to flow continuously upward through sand, and his analysis of the effluent showed a considerable liquefaction of the suspended

solids as well as a reduction in the total quantity of organic matter. But as the cause of such action was not understood (though suggested in the description of the Automatic Scavenger), and the effluent so much fouler than the fresh sewage, and still in need of treatment, little interest was aroused at the time.

## SCOTT-MONCRIEFF METHOD.

In 1891, Mr. Scott-Moncrieff constructed at Ashtead, in England, a bacterial tank, in which the crude sewage was admitted from below and made to pass gradually upward through a bed of stones. The reports of the chemists who examined it showed it to be capable of liquefying the solids of the sewage passing through it, and of causing incidentally a disappearance of a portion of this in the form of gases. The effluent from this tank, of course, contained a large quantity of matter in solution, but in a condition which rendered it readily oxidizable. By alternating periods of rest and use, Mr. Moncrieff tried to complete the decomposition by bringing about the oxidation of the result of the anaerobic liquefaction, but without much success. Later, after it became evident that the preliminary anaerobic action and the subsequent oxidation were entirely separate processes, he perfected his system by causing the effluent to pass through layers of coke placed vertically over one another, with intervals of about three inches between. The layers are nine inches thick, and the coke is broken to a diameter of one inch. By this means the bacteria corresponding to the different phases of the oxidation processes are kept separate from each other, and a remarkable degree of purification is the result.

The system is said to work satisfactorily at rates of from 500,000 to 1,000,000 gallons per acre of coke bed per twenty-four hours. The cost of construction must be considerable, and obviously it requires protection from cold, which would add to the expense and prove a serious objection to its use in many places.

## CONTACT BEDS.

Another bacterial system which includes the destruction of the bulk of the sludge is known as the Bacterial or Contact Bed System. This method was developed by W. J. Dibdin, while conducting experiments for the London County Council, the town of Sutton, and other places. The average flow of sewage at Sutton is about 500,000 gallons per day, and chemical precipitation with subsequent land treatment was the method of disposal employed. This proving unsatisfactory, Mr. Dibdin was engaged in 1896 to advise with regard to the construction of a bacterial system. The methods suggested by him have proved successful there, as

they have since in several other instances. In the short description which follows, the plates and photographs of the Manchester experimental plant will be used to illustrate the method of working. The materials used in constructing the tank as well as that used in filling it, vary considerably, but their mode of action is the same.

The beds are constructed and worked in pairs in the Double Contact System, the sewage passing first through one and then the other, which must thus be at a lower level. The rate of inflow and outflow is controlled by valves, sometimes worked automatically.

The depth of the filtering material is in each case usually about three and a half feet. In depressions in the bottom of the tank are perforated pipes for draining off the filtrate, which, by an arrangement of the outlet, remain full after the bed is emptied. Immediately around these pipes coarse material is placed, but the material in the body of the bed is usually of uniform size. The upper of the two beds is the coarser. The general arrangement is shown in Diagram 1.

The sewage is first to be passed through a revolving self-cleaning wire screen, which removes rags, paper, etc., and the "grosser solids." It should, especially if from a combined system, be also made to pass through a small detritus or sand tank, which intercepts heavy matters, such as sand, gravel, etc., which do not require treatment, and would choke the bed. It is then run on to the first or coarse bed, called sometimes the "roughing" filter. The method of distributing the sewage uniformly over the surface of this bed is a matter of considerable importance in the case of large beds. When the bed is filled to within a few inches of the surface, it is held so for two or three hours, or for a time depending upon the rate at which the system is to be worked. This period of "resting full," during which the sewage is in "contact" with the surfaces of the filling material, gives the system its name.

The outlet valves are then opened, and the bed is emptied at any desired rate. It is then allowed to rest empty for a definite period, at the end of which the process is again repeated. The time of a complete cycle thus includes the time of filling, resting full, emptying, and resting empty, and depends upon the number of complete cycles to be made in twenty-four hours. This is usually not more than three or four at most. At regular intervals of a week or longer the beds are left empty for a whole day. The process taking place in this coarse filter, especially when standing full, is largely anaerobic, as shown by chemical analysis of the effluent. The sludge disappears in the beds, and does not accumulate, being broken down and liquified by bacterial action. The completion of the process,—the oxidation of the effluent from the coarse filter, is accomplished by the second or finer bed. When the coarse bed is

emptied, the effluent is run into the fine one, which is worked in the same way as the first. The organic matter being in a suitable condition, the process here is chiefly oxidation by the aerobic bacteria.

The effluent from this bed has a good appearance, is free from all objectionable odour, and keeps perfectly sweet in open or closed vessels.

The percentage purification shown in the Sutton experiments is given by Dr. Rideal as follows:—

| | Oxygen Absorbed. | Free Ammonia. | Albuminoid Ammonia. | Suspended Matter. |
|---|---|---|---|---|
| By the coarse bed.. | 53 | 69 | 47 | 94.0 |
| By the fine bed.... | 29 | 21 | 25 | 4.4 |
| Total purification. | 82 | 90 | 72 | 98.4 |

A third bed of still finer material may be added, if a higher degree of purification is desirable, which will rarely be the case.

The filtering material may be coke, clinker, broken stone, broken bricks, burnt clay, slag, ashes, etc., according to the cost, which will vary with the locality. Of these coke seems to have some slight advantage over the others, and is more generally used. At Lichfield coal has been used, with good results. In fact, one of the great advantages of the system is, that material that will serve the purpose can be found in some shape or other in almost every locality. The construction of the tanks need not be expensive. The sides may be sloping, which favours cheap construction. In any case, the beds are usually quite shallow. Experiments have been made with filters as deep as 13 feet, but the tendency seems to be towards three and a half feet beds, and several cycles in the course of 24 hours. The tanks will usually be built of concrete or brick, though one of the Dibdin tanks at Sutton was made in heavy clay by digging a pit about three feet deep, burning the excavated clay, and using it for the filtering material to fill the pit with, after the collecting drains for the effluent had been put in place.

This system requires a fall somewhat greater than the combined depths of the beds, which may in some cases necessitate pumping. The preliminary digesting bed of the Scott-Moncrieff system, on the contrary, requires only the very slight fall necessary to overcome the resistance to the flow through the beds.

The process is not completely anaerobic at any stage, hence the necessity for screening, and also the tendency to loss of capacity from the accumulations among the particles of the bed of fragments of straw, chaff, etc. This reduction of holding capacity proceeds

rather rapidly at first, but seems in many (but not in all) cases to finally reach a maximum which is permanent. The time occupied in filling, resting full, etc., the length of the cycle, the length of the interval between the lengthened periods of rest and aeration, vary with the quantity of sewage, the rate of flow, material, etc. The system is already in use in several places, and is working satisfactorily.

Whether such materials as burnt clay, coke, etc., will be permanently effective, and not disintegrate, is yet to be proved.

## SEPTIC TANK.

The invention of this system is claimed by Mr. Donald Cameron, city surveyor of Exeter, England. A plant built by him for the disposal of a portion of the sewage of that city has been in operation since 1895. Diagram 3 shows a sketch plan, and Photograph 3 a view of the works.

In this system, as in Moncrieff's, the purely anaerobic part of the decomposition process is kept quite separate from the aerobic.

After passing through a small "grit chamber," which intercepts sand, gravel, etc., the sewage, without further preparation, enters one end of a long, narrow tank, seven or eight feet deep, through which it flows very slowly, finally escaping at the other. In the Exeter plant, this tank is 56 feet 10 inches long, 18 feet wide and 7 feet 6 inches deep. The grit chamber is 7 feet long, the same width as the tank, and 10 feet deep. The sewage flows from this chamber into the tank over a bridge wall one foot below the level of the sewage which is kept constant. The outlet consists of a horizontal slotted pipe extending the width of the tank, and also situated a foot below the surface. The object of this relation of inlet and outlet to depth and width of tank, is to cause as little disturbance as possible to the contents of the tank, and to produce a uniform flow in all parts of it. The rate of flow will be very slight, not averaging as a rule more than one inch per minute. A certain amount of mixing which takes place has the effect of rendering the sewage which escapes from the tank more uniform in quality than when it arrives.

This tank is built of concrete, and is arched over and covered with turf to exclude light and air. The conditions are thus essentially anaerobic, and the process, which has probably already begun in the outfall sewer, is carried on under the best possible circumstances. It does not reach its full intensity for some weeks after being put into use, during which time the bacteria are developing and establishing themselves. The changes which take place are,

according to Dr. Rideal, of the nature of the hydrolytic ones already referred to, and indicated by the inner path on the diagram (A), connecting "Fresh Organic Matter" with "Free Ammonia, etc."

In the Exeter tank there is a glass inspection well, from which the action going on in the tank may be observed. When the sewage enters the tank any light fatty matter it contains rises to the surface, while the heavier sludge falls to the bottom. Here there is great bacterial activity. Anaerobic bacteria begin to break down and liquify the solids, liberating various gases, which, appearing as bubbles in the body and on the surface of the masses of faeces, etc., cause them to float to the surface, where they soon form a leathery-looking scum from two to six inches thick, in which the processes of decomposition and liquefaction go on with undiminished activity. The greater part of the solid matter is thus thrown into solution during the slow progress through the tank, but a considerable portion is converted into gas, the bubbles of which, continually rising and carrying up bits of solids from below, keep up a constant mixing of the contents of the tank.

The location of the inlet and outlet prevents the layers on the surface and bottom from being disturbed. The effluent, which is drawn from between them is dark in colour and offensive in odour. It contains more matter in solution, but has lost on the whole a considerable percentage of organic matter,—chiefly matter in suspension. The percentage purification is given by Rideal as 29, measured by oxygen absorbed, and 46 by organic nitrogen removed. Corresponding figures from Dr. Dupre, chemist to the Medical Schools, Westminster, are 63.5 and 27.7. As we have seen, much of this has been converted into gas, largely methane or marsh gas, ($CH_4$), which renders it inflammable. This is collected at Exeter and used for lighting about the works.

The active decomposition going on at the bottom of the tank prevents the accumulation there of much organic matter. After being in operation for three years, the deposit at the bottom of the Exeter tank had not accumulated to such an extent as to make its removal worth while. When this becomes necessary, it is drawn off through a sludge pipe under the bottom. It is a dark, peaty-looking substance, about one-third organic. It may be used for filling, as the smell is not disagreeable, and it is to outward appearance, very much like ordinary prairie soil.

The surface layer at first increases in thickness, owing to the accumulation of the more resistant materials; but they also finally succumb to the intense bacterial action, and, after that, the depth of this scum, though fluctuating somewhat, remains fairly constant. It is an interesting fact that full bacterial activity may be set up in a new septic tank in a very much shorter time if it is "inoculated" with a portion of the scum from a working tank.

As has been said already, the effluent from this tank has already been considerably purified. The total amount of organic matter has been greatly reduced, and what remains is largely in solution, and in a condition which renders it very easily oxidized. Hence, in some cases, it may be turned at once without further treatment into a body of water. But usually further treatment will be required, and this is provided for in the system by contact beds, into which the effluent from the tank is run after passing over aerating weirs. These beds of the Exeter plant, shown in Diagram and Photograph 3, are each 36 feet long by 20 feet wide, and filled to a depth of 4½ feet with crushed furnace clinker, resting on 6 inches of coarse gravel. In filter No. 3 coke breeze is used instead of clinker. The filtrate is collected by a system of underdrains. The flow through the septic tank is continuous, but the filters are worked in much the same way as the contact beds already described; that is, they are first filled, then held full for a time, then emptied, and after resting empty for a time have the same process repeated. In the Exeter plant the opening and closing of the necessary valves is effected by an automatic arrangement, which causes the beds to act in rotation, thus securing great regularity with very little attention. Some such device is necessary in any system of beds operated in this way.

In these aerobic beds the products of the fermentation which has taken place in the septic tank are acted upon in the presence of oxygen by aerobic bacteria, and a very high degree of oxidation takes place. The total percentage purification effected by tank and filter, as shown by analysis, is as follows:—

| Authority. | Albuminoid Ammonia. | Oxygen Absorbed. |
|---|---|---|
| Dibdin and Thucidum | 63.2 | 80.9 |
| Dupré | 84.9 | 88.3 |
| Pearman and Moor | 80.0 | 90.0 |
| Perkins | 64.4 | 78.7 |
| Rideal | 77.0 | 82.0 |

The effluent is clear in appearance, with very little colour, and without any offensive odour. It keeps well without showing any tendency to putrefy, and is said to be quite as satisfactory as that obtained from well-managed sewage farms. An advantage of the system is that there is almost no loss of head in the tank, and that therefore the only fall necessary is that required by the beds.

The three processes described above are the most important of those which separate the process into an anaerobic and an aerobic portion. The order of the operations is shown by the inner path on the left side of diagram (A), the "partial aeration" step corresponding to the changes between the completely anaerobic and oxidizing stages.

There are also the Lowcock, Waring and Ducat processes, which are chiefly aerobic at all stages. The object of these methods is, by bringing large quantities of air in contact with sewage by mechanical means, to enable the filters to be worked continually.

## LOWCOCK'S SYSTEM.

In the Lowcock system the beds consist of a bottom layer of coarse coke, a body of coke breeze, and a top layer of broken stone and sand. The air enters at a low pressure through the outlets, which are always kept open. In the plants in operation at present, however, daily periods of rest are said to be found necessary as well as preliminary sedimentation.

## DUCAT'S SYSTEM.

In Col. Ducat's system the beds are composed of layers of ¼ to ½ inch vitrified clinker, alternating with thin layers of coarse pebbles. The walls are composed of short lengths of 6 inch drain pipes set in mortar, with an upward tilt towards the outside to prevent the escape of the sewage as it trickles through the bed, and at the same time to provide for its aeration.

## WARING'S SYSTEM.

The method devised by the late Col. Waring at Newport consists of a screening tank of broken stone, through which the sewage sinks in one part of the filter and rises in the other, after passing under a dividing wall. This bed removes the coarser suspended matter, which eventually clogs it. It is then thrown out of use, and a current of air from a blower is made to pass upward through it, and maintained for a few days, until the accumulated solids disappear, as they are said to do, under bacterial action. The effluent from these "strainers," as they are called, is run on to beds of crushed coke, covered with sand, through which it passes downward, meeting air currents, which are forced in through the collecting pipes which cover the floor. The system is in operation at Willow Grove Park, Philadelphia, where it is said to work satisfactorily, the effluent being used to sprinkle the roads and lawns in the park and the neighbouring residence avenues.

With these methods, besides the cost of artificial aeration, protection from cold is necessary in winter, which adds to the expense.

## MANCHESTER EXPERIMENTS.

About the middle of 1898 the Council of the City of Manchester, England, engaged Messrs. Baldwin Latham, Percy Frankland and

W. H. Perkin, Jun., three of the most distinguished experts on the subject in England, to advise them as to the choice of a new system of sewage disposal for the city. The dry weather flow of sewage amounted to 30,000,000 gallons per day, and the method of disposing of it was (as now) by chemical precipitation and partial land treatment. The effluent from the precipitation tanks was discharged into the Ship Canal, but the purification was so inadequate that the fouling of the canal water was the cause of frequent complaints, which finally led to the employment of the board of experts mentioned above. In making their investigations and reports they had the advantage of the experience of a great many plants in operation, including most of the new bacterial systems described above. After visiting and examining many of these plants, and carefully considering various processes, the board decided that the most promising method of disposal would be by one of these bacterial processes. And to verify this view, and at the same time to indicate which would prove most suitable for Manchester, they undertook a series of experiments, 1st, with bacteria or contact beds; 2nd, with the septic tank system, and 3rd, with the Roscoe filters. As it was a combination of the first two methods that was finally recommended, a short description of the plants used, and some of the conclusions arrived at, will now be given.

Diagram and Photograph 1 show the contact bed plant. The tanks, situated as shown, were constructed of concrete 6 inches thick, and plastered with half an inch of cement mortar. They are 33 feet 6 inches square on top, and 17 feet 6 inches on the bottom, and 4 feet deep. The bottoms are channelled to receive the 6-inch and 2-inch perforated pipes for drawing off the filtrate. The filtering medium is composed of clinkers laid to a depth of 3 feet. With the exception of the rough material immediately surrounding the pipes, the size of the clinkers is uniform throughout the bed. Five beds were used, the material being of a different size in each bed. In the finest it was from ⅛ inch to ½ inch; in the coarsest from 1 inch to 3 inches.

These beds were worked in pairs, according to the method already described, of periods of filling, resting full and emptying. They were worked with raw sewage, sewage settled in a comparatively small tank, and sewage which had undergone anaerobic action in one of the large open precipitation tanks.

The septic tank plant is shown in Diagram and Photograph 2. The tank is constructed of concrete plastered on the inside with cement mortar. It is 40 feet long, 12 feet wide, and 9 feet 2 inches high, having an arched roof, in which air-tight manhole covers are placed for inspection. There is a grit chamber, and the effluent passes in thin sheets over the sides of an aerating trough, before it goes to the filter beds, which are six in number, and of the average

area of 294 square feet. Two-inch and four-inch butt-jointed perforated drains are laid on the bottom to collect the filtrate.

The filtering material is 4 feet deep, and is composed as follows from the bottom upward:—

1 foot in depth of clinker, 1 inch to 3 inches.

2 feet 9 inches screened clinker, ⅛ inch to ¾ inch.

3 inches of residue from above, which will pass a ⅛ inch mesh.

These beds are filled and discharged automatically.

It was found in the course of the experiments, as it has in other places, that the open septic tanks produced results equal to those of the closed tanks; the amount of air or light which can be absorbed when the scum covers the tank not affecting the action. The mixing of the sewage in the tanks, which renders it more uniform in character is regarded as being of great importance in the further purification in the beds.

The system finally recommended by the experts, was treatment by an open septic tank and two contact beds. A satisfactory effluent can be obtained in this way at the rate of about 700,000 gallons per acre per day. The sewage of Manchester contains an unusually large proportion of trade wastes, which it was at first thought would render bacterial treatment unsuitable, but the results of these experiments have removed all doubts as to its capability in this respect.

The plant, which is to deal with a daily dry weather flow of 30,000,000 gallons is recommended to consist of 60 acres of contact beds, which are thus to be capable of purifying at the rate of 500,000 gallons per acre per day, allowing one day a week for rest. The system is also to be capable of dealing with a storm water flow of 90,000,000 gallons per day. This is effected by storing the first flush of storm water for regular treatment, since it is almost as foul as the sewage; and then disposing of the rest, which will be greatly diluted, by an accelerated bacterial treatment of short double contacts, or, where the dilution is great, by single contacts. Experiments were made with the storm water to show this to be possible; and an additional area of 25 acres was estimated to be necessary for the purpose. The material recommended for the contact beds is ½ inch to 1½ inch clinker.

The experts express their confidence in the capability of this method to produce an effluent which will not only not pollute the canal water, but will materially improve its condition; and they further expect that the saving in the expense of operating the system when compared with that necessary in connection with the present plant, will offset to a very considerable extent the cost of constructing the present works.

The data in regard to the cost of construction of bacterial systems, is as yet necessarily rather limited. In any case, it would

be impossible to give any statement or estimate of cost which would be generally applicable. The cost of constructing the beds and tanks will depend upon the price of the materials used, and the nature of the site available. But in comparing such systems with sewage farms or intermittent filter beds, there are other things to be taken into account besides the first cost of the plant itself. The great area of land required for the latter methods necessitate their location at a considerable distance, and as this land must also be of special quality, pumping will often be required. With bacterial plants, on the contrary, the choice of a location is not limited by the necessity of finding proper material, the area required is relatively small, and their operation, especially when the tanks are covered, does not cause a nuisance even in the immediate neighbourhood. They may therefore be placed to suit the natural outfall of the system, and usually so as to receive the sewage by gravitation. It sometimes happens that a partial purification only is all that is necessary, such as the removal of sludge and floating material. In such cases treatment in the septic tank alone will be sufficient. For these and similar reasons the adoption of this method may result in a considerable saving in the expense of constructing long outfall sewers, pumping plants, etc., which should be credited to it when making comparative estimates of cost.

The operating expenses will certainly be less than for any of the artificial processes, and will probably not be greater than for land treatment.

In conclusion, it may be stated that the experience of the last four or five years seems to indicate that the main problems in the disposal of sewage have been solved; and that further progress will be in the direction of improvements in the methods of applying the principles, and in the details of operation.

Whether the preliminary liquefaction, which is the distinguishing feature of these new bacterial methods, is essentially anaerobic or not is not certain, in view of the action which takes place in the coarse bed of the Dibdin system. In any case preliminary treatment in an open or closed septic tank seems advisable.

A system of this kind is especially adapted for isolated plants such as for large country houses, hospitals, schools, etc., being compact in arrangement, and, when covered, free from nuisance, while at the same time protected from the effects of cold.

The degree of purification effected by such methods has been already referred to, and the effluents have been shown to be entirely satisfactory, both from the point of view of their chemical purity, and of their freedom from putrescible material. The bacterial efficiency is, however, very low, the number of bacteria in the effluent not being very greatly less than in the raw sewage. This

is, of course, an advantage rather than an objection if the discharge is into a stream not used lower down as a water supply; since the bacteria will tend to bring about the further oxidation of any organic matter which may still remain in the sewage. The possible presence of pathogenic organisms among the rest would seem to constitute a danger if the water were subsequently used for drinking. The opinion of those who have investigated this subject seems however, to prove that the conditions in the sceptic tank and filter beds are decidedly antagonistic to the existence of the typhoid germ, for instance, and that the chances of pathogenic bacteria surviving the passage through tank and beds are small. Dr Rideal has suggested sterilizing the effluent, in cases where it is necessary, by chemical means.

In considering this matter, however, it must be remembered that the object of purifying sewage is not to produce a drinking water. Such a result is, of course, often desirable if it can be obtained without undue expense. But when a community has purified its sewage by the methods just described, before discharging it into a water course, it would seem as if all reasonable precautions had been taken, and its duty in that respect had been fulfilled. If the water is subsequently required for a domestic supply, it should be the part of the community so using it to provide any further guarantee as to its wholesomeness and freedom from danger. Such a mutual arrangement between places thus situated with reference to each other would ensure greater efficiency, more satisfactory results, and at the same time an equitable apportionment of the cost of the necessary sanitary works.

## DISCUSSION.

Mr. W. M. Davis

The paper by Mr. Lea on Sewage Disposal is a concise history of bacterial treatment of sewage up to the present time, written in such a clear, logical manner as to make it very interesting ; the graphic illustration of changes in living and dead organic matter is very ingenious.

Mr. Lea makes the statement that the new bacterial methods solve the problem of sludge disposal ; it appears, however, to the writer that this is going rather farther than the facts would justify. The experience of places where these methods have been tried is that about 20 per cent. of the suspended matter in sewage is deposited as sludge ; this sludge is composed chiefly of inorganic matter and is inoffensive in character, but still provision must be made for its removal.

In the contact bed system lack of permanency must be a serious objection ; the clogging is by inorganic matter, and prolonged aeration does not relieve it ; preliminary sedimentation would apparently remedy this.

The septic tanks constructed at Urbana and Champaign, Ills., by Prof. Talbot were installed about the same time as those built by Donald Cameron in England, the chief difference being in the capacity, the English tanks being about twelve times the size of the American tanks. It would be interesting to know whether the difference in the character of the sewage necessitates so great a difference in the capacity of the tanks.

The presence in tank effluents of elements, which are absolutely necessary to the growth of the plants, would suggest the economy of utilizing the effluent where practicable for the irrigation of a sewage farm. It is probable that the revenue from the farm would materially reduce the expense of management of the disposal plant.

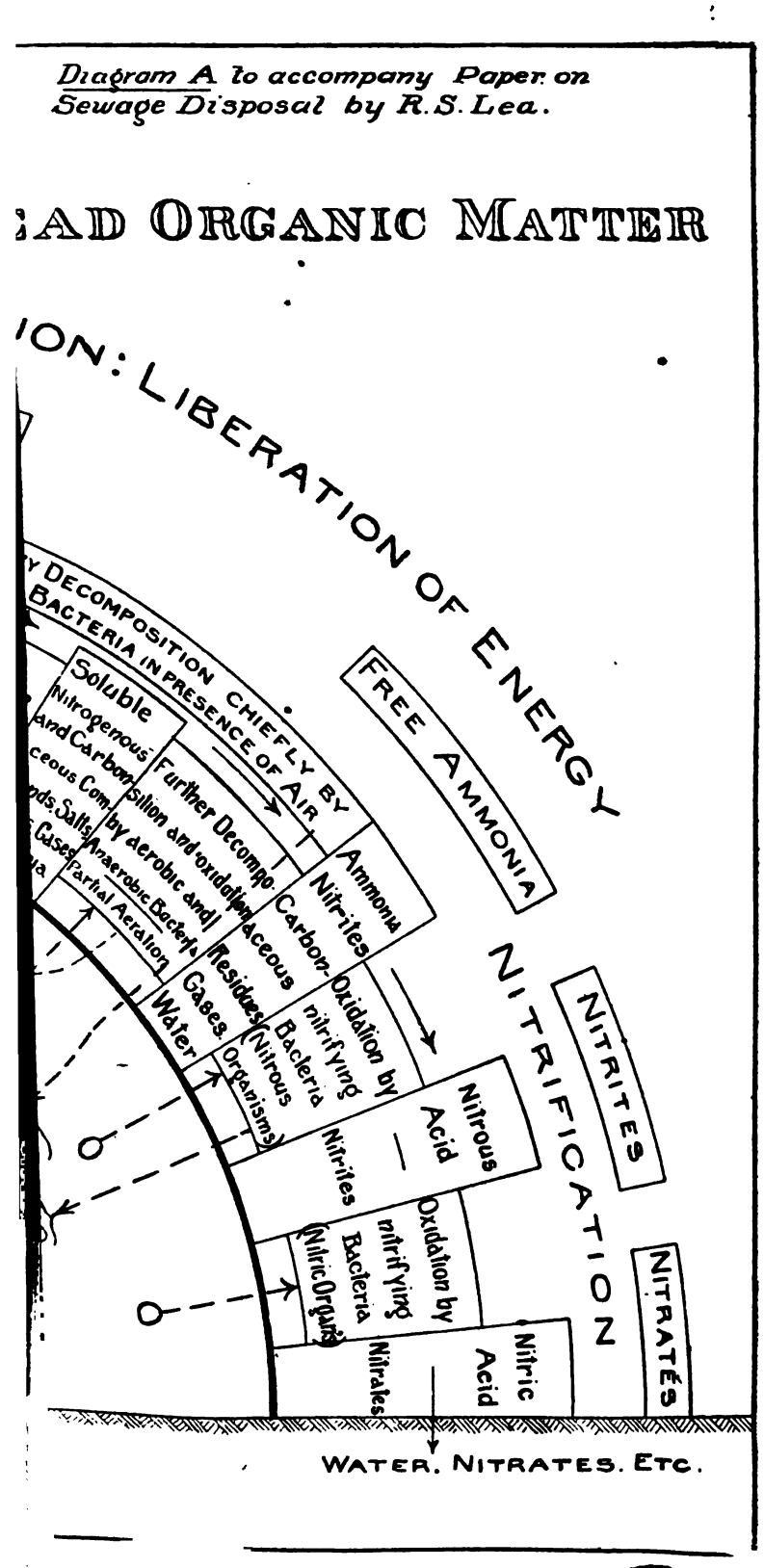
Mr. R S. Lea

Mr. Lea in reply to Mr. Davis said his paper was read from the manuscript in April, 1900, and hence no proofs were sent out in advance as is customary. As published in the Transactions of the Society, it is considerably extended, and some slight modifications have been made in accordance with further experience in the working of bacterial methods of disposal.

Regarding the statement referred to by Mr. Davis, that these methods solve the problem of sewage disposal, what the author means is that the utilization of the action of the anaerobic bacteria as a prelimary to the subsequent oxidizing processes furnishes the true solution of the problem. There is, of course, still much to learn regarding the details of the process under different circumstances, but, if the results obtained by Prof. Talbot in the Champaign tank, which Mr. Davis refers to, could be depended upon in all cases, the chief difficulties in the disposal of sludge would be removed.

There is no doubt that the proper capacity and method of working the septic tank depends to a great extent upon the character of the sewage, and it is not reasonable to expect satisfactory results unless this is taken into account. In any case the sewage should remain in the tank ten or twelve hours in order that septic action should take place.

*Diagram A to accompany Paper on Sewage Disposal by R.S. Lea.*

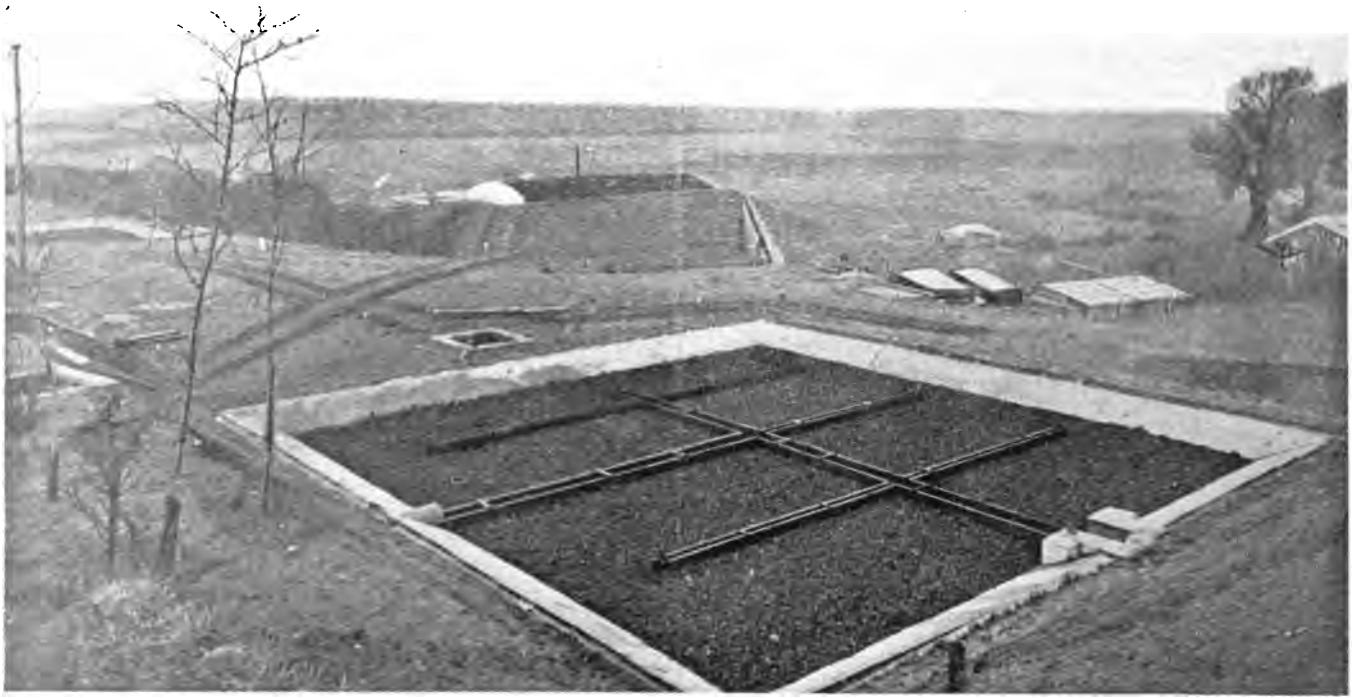



GENERAL VIEW SHEWING BACTERIA BEDS A AND B. MANCHESTER (ENG.) EXPERIMENTS.

Mr. W. M.
Davis

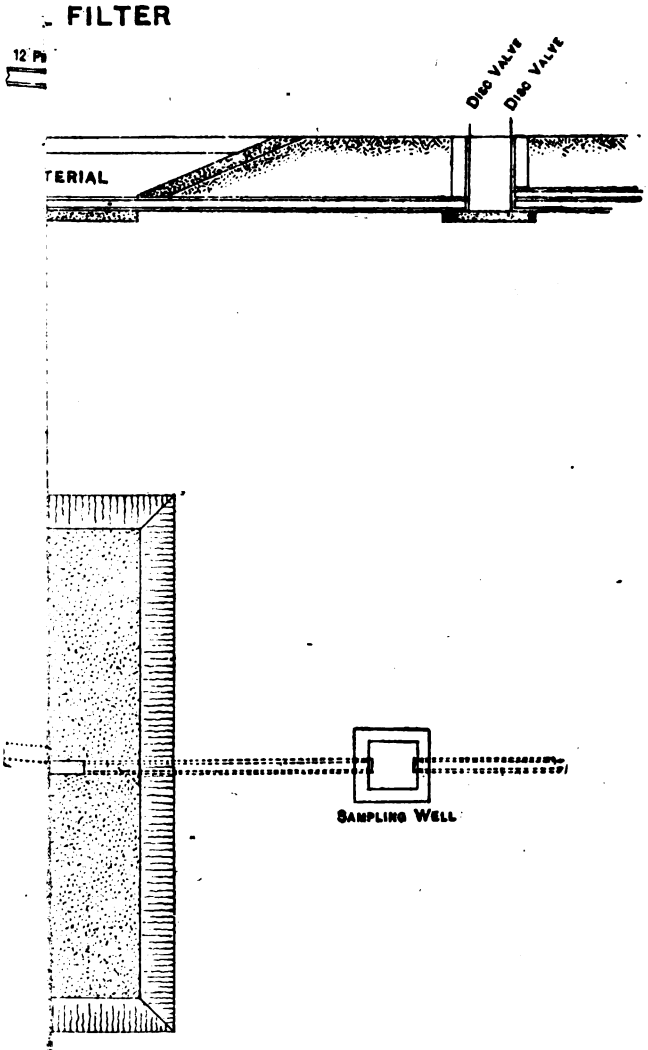
Mr. R S. I

Photograph 1



GENERAL VIEW SHEWING BACTERIA BEDS A AND B. MANCHESTER (ENG.) EXPERIMENTS.

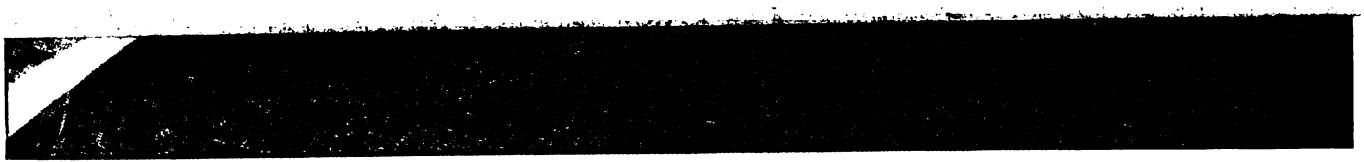
**Diagram 1**



VIEW SHEWING SEPTIC INSTALLATION. MANCHESTER (ENG.) EXPERIMENTS.

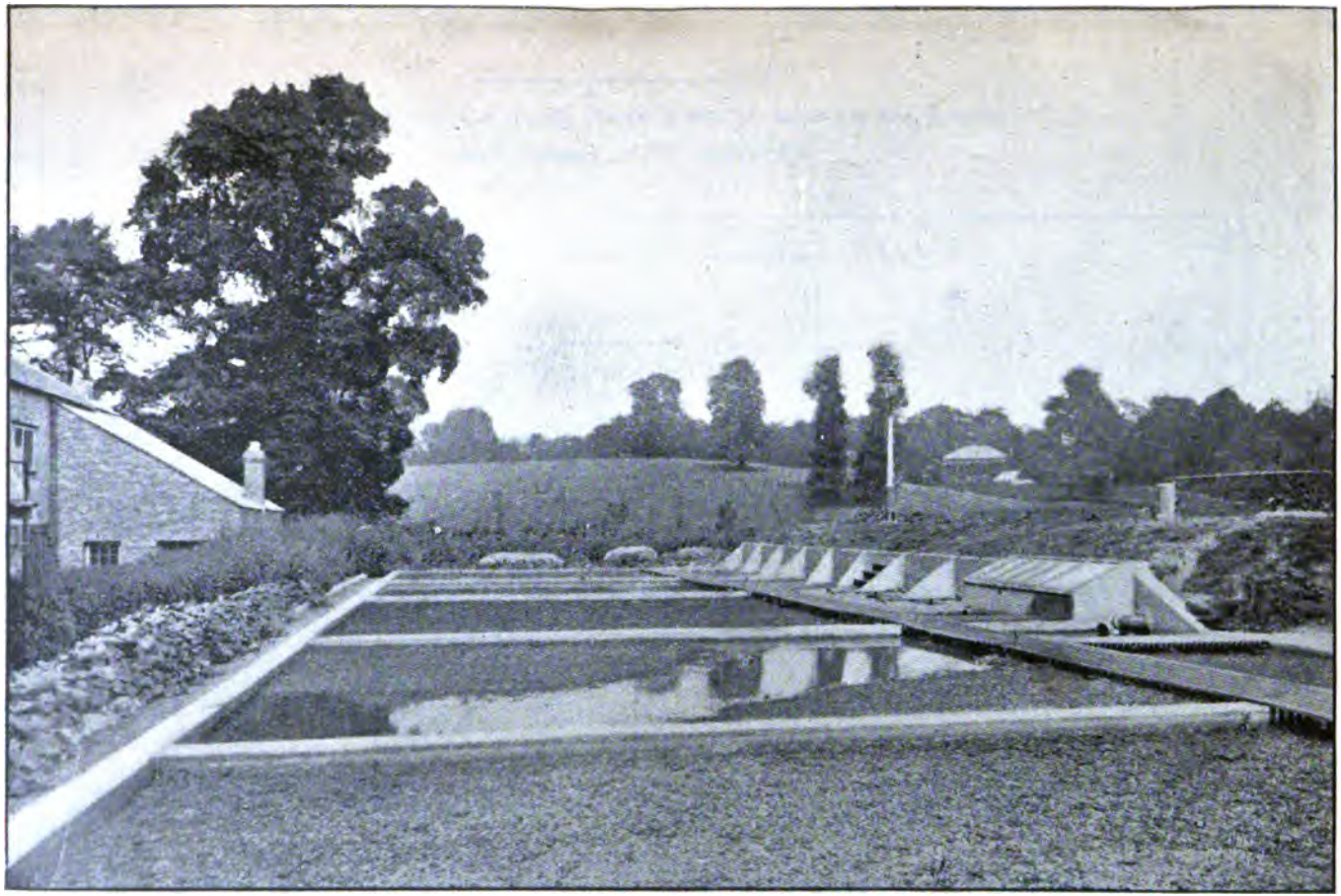
**Photograph 2**

VIEW SHEWING SEPTIC INSTALLATION. MANCHESTER (ENG.) EXPERIMENTS.



EXETER (BELLE ISLE) INSTALLATION. (SEPTIC TANK SYSTEM.)

Photograph 3



EXETER (BELLE ISLE) INSTALLATION. (SEPTIC TANK SYSTEM.)

Thursday, 31st May.

G. H. DUGGAN, Vice-President, in the Chair.

Donations to the Library were reported as follows:

Forty-six volumes, Report of Chief of Engineers, U. S. Army, from Mr. P. W. St. George, M.Can.Soc.C.E.

"Induction Coils and Coil-Making," from Mr. H. L. Shepherd,

The following letter, addressed to Dr. Bovey, was read:—

"Dear Sir,—

Your letter of May 23rd, addressed to W. H. Wiley, at hand. In reply, we mail you this day our catalogue. Please mark to the amount of $50 the books you would like, except those that are starred, and we will send them.

Hoping this is satisfactory, we are,

Yours truly,

JOHN WILEY & SONS."

On motion of Mr. L. Skaife, seconded by Mr. G. H. Duggan, it was resolved: "That the thanks of the Society be transmitted to Messrs. Wiley & Sons for their gift to the Library."

Messrs. W. C. Thomson, E. S. M. Lovelace and S. F. Rutherford, having been appointed scrutineers of the ballot, reported the following as elected:—

MEMBER.

CHAS. FERGIE.

ASSOCIATE MEMBERS.

W. E. MANN, G. STEAD.

Transferred from the class of Associate member to the class of Member:

W. CHASE THOMSON.

Transferred from the class of Student to the class of Associate Member:

E. LOIGNON, W. A. MACDONALD,
A. J. MACDOUGAL, J. L. TIGHE.

STUDENTS.

D. M. CAMPBELL, J. A. CAMPBELL,
L. B. CHUBBUCK, G. G. GRUNDY,
E. P. JOHNSON.

*Paper No. 151.*

# ENGINEERING RECORDS IN THE RAILWAY OPERATING DEPARTMENT.

By C. E. Cartwright, M. Can. Soc. C. E.

When, on completion of construction, a railway is turned over to the operating department, it is generally found that the maps, profiles and other records are far less complete than generally supposed. Even with very careful location, the unevenness of the ground, obstructions from trees and changes made during construction, introduce inaccuracies in distance, breaks in chainage caused by changes of alignment increase or decrease length of line and are difficult to keep account of.

The profiles also have been made with differing datum and changes of grade made at the last moment, are often unrecorded.

Sidetracks, buildings, water tanks, etc., are seldom definitely located until construction is nearly complete, and not generally until after the general construction work is finished.

Altogether, even at the best, the plans and records are more or less deficient.

Sometimes, also, a railway company acquires an old line from another company, or several short lines are amalgamated; in these cases it is often found that the records are almost altogether lacking, or are in a very incomplete and unreliable condition. Often the engineer in charge will find that he will have to get along the best way he can with the information available, making special surveys as the necessity arises; these surveys will be made in a hurry for some special purpose and will be of little or uncertain value for other uses. Surveys of this kind accumulate, made by different parties for different purposes, with varying degrees of accuracy, and after a time it is seldom known how much reliance can be placed in them; in consequence it is often necessary to revisit the ground and make new surveys; a great amount of work has to be done, which would have been unnnecessary if reliable plans existed.

The remedy for this state of affairs is a complete re-survey of the whole road, especially when it is an important road in a thickly settled country, with towns and villages at short intervals.

On a railway in the Central States, with which the writer was employed, it was decided to make a complete re-survey, the line having been recently acquired from another company; the few existing plans were incomplete and disconnected, a great amount of uncertainty existed as to the company's title to right of way and other property, and it was also proposed to expend a considerable sum in improving alignment and grades, accurate plans and profiles being needed for this purpose.

The railway was first carefully measured from end to end, starting with zero at one terminus; a 100-foot steel tape was used; each hundred-foot station was marked with white paint on the inside of the rail; every tenth station was referenced by an oak stake, 3 inches square, set 7½ feet from the centre line; stakes were also set at every mile, to be afterwards replaced by standard milé posts. After the measurement was completed, the line was gone over by the transit party. This party made a traverse of the line, not stopping to run tangents to intersection and put in curves, simply getting a record of the centre line of the track, as they found it on the ground. On tangents, a sight would be taken on the track ahead as far as visible, and a straight line run, any deflection in the track being noted.

The intersection of all township, section, quarter-section and property lines were obtained, the angles recorded, and distances measured to the nearest section or quarter-section corners, one member of the party being employed most of his time in looking up monuments.

Plusses to points of intersection were obtained from the stations marked on rails by the measuring party.

A record was made of the fences on each side of the right of way, and distance from the centre line, this being often important as a means of determining a disputed boundary, where the fence had been in existence for a long period, as many deeds did not state the width of the right of way.

In villages and towns, the streets and lots adjacent to the company's property were located; all important factories, with the tracks leading to them, even if on a foreign railroad, and all sidings and structures on the company's property, particular attention being given to apparent encroachments, it being often found that buildings were wholly or partially on the company's property without any lease having been made. In making a survey through a village or town, the transit party was furnished with copies of the official plates, previously obtained at the county seat, to aid them in locating lines and streets.

The transit party measured all bridges, buildings, culverts and other structures, located all "Y" and railroad crossings, and public and private road crossings.

The level party followed, taking levels at every hundred-foot station, on top of tie, at ends of bridges, on railway crossings, of level of water in streams, and approximate levels of adjacent ground. Check levels were run and bench marks established at about half mile intervals, and oftener at places likely to be needed. Levels were connected with sea levels taken from United States Government surveys. The plans were drawn on white drawing paper in sheets, on a scale of 400 feet to an inch, each sheet showing the line across a square mile section of land, a whole section or two adjoining half-sections being shown on the sheet.

The top of sheet was north in every case, all distance and angles obtained on the ground to section and property lines were recorded on the sheets.

On top of each sheet was a plain title giving number of section, township and range.

All deeds and agreements were carefully gone over and compared with the plans, right of way coloured in red, with name of grantor, page, and number of record book, and any conditions in deeds noted on plans. Villages and towns, where the scale of 400 feet to an inch did not allow sufficient detail to be shown, were drawn also on a scale of 100 feet to an inch, a large town often requiring several sheets, the same ground being covered, with less detail, on the smaller scale.

The sheets, when completed, were numbered and bound together by counties, the first page being devoted to title and the second to an index map of the county, showing the route of the railway.

The centre line was drawn in red ink, all station numbers and plusses being also in red; distances and all lettering were shown in black.

Before binding, all the plans were copied on tracing linen.

The profiles were drawn on the usual scales of 400 feet to an inch horizontal, and 30 feet to an inch vertical.

In cases where the engineer is unable to have a complete resurvey made, it will be advisable, as time permits, to make accurate surveys of all yards and station grounds, depending for general details of alignment outside these limits on the existing right of way maps.

If an accurate set of yard and station ground plans are obtained to start with, it will be a comparatively easy matter to keep them correct as changes are made.

A statement should be prepared, giving length of all sidings, spurs and "Ys," made from actual measurements, and not from foremen's reports. The form can be made with several blank columns to be filled in from time to time, with "Track laid during ——," "Track taken up during ——," "Total length on ——." This state-

ment will show at a glance length of track on any siding, and avoid a search on plans and profiles for the information.

A chart, showing graphically the different makes, weights and date when laid, of the rails in use should be made, and corrected as new rails are laid.

A bridge book should be kept, devoting a page to each bridge or trestle, giving style, spans, size of stringers, when built, when repaired or rebuilt, conditions when inspected, etc.

A record should be kept of all leases of the company's property, a copy of lease and plat filed.

Detailed statements should be kept of the actual cost of all structures built, and, on completion, plans made showing the structure actually as built, showing depth and character of foundations of bridges, retaining walls, etc., and all differences from the original designs. A condensed plan and profile may be prepared when time permits, showing a great amount of general information, useful in the track and operating departments. The scale will depend somewhat on the length of line it is desired to represent, but, even on a scale as small as one mile to an inch horizontal, and 100 feet to an inch vertical, the principal grades, sections, mileage, water tanks, sidings, railway crossings, etc., can be shown.

The plan will probably have to be somewhat distorted in order to keep it on same paper as the profile, and lengths of sidings and size of structures exaggerated. The arrangement can be somewhat as follows:

On top a series of lines showing number of telegraph wires; lines showing the fencing, the mileage and the track sections; below this, a plan or graphical chart of the line, showing general geography and alignment, degree of each curve being shown by figures. Then the profile, showing bridges, grades, stations, etc.; below the profile a series of lines showing rails, joints, ballast, and new rails, new joints, new ballast.

An important matter is a system of filing plans so they can be quickly found when needed. When blue prints are much used, it is best to file the tracings. When plans are numerous a card index will be found most convenient. A plan can be indexed on the cards under several headings, and new plans can be added and changes made without spoiling the index.

## DISCUSSION.

Mr. W. T. Jennings

Mr. Cartwright's paper on "Engineering Records in the Railway Operating Department" contains numerous good suggesstions to young engineers or those untrained in or unaccustomed to the usages of a large and well regulated engineering office.

The Railway Act of Canada contains sections clearly defining the records, plans and profiles required for registration and other purposes in this country, and which largely cover the useful requirements in this branch of engineering.

The lack of method, through want of proper apprenticeship or training, is one of the conspicuous features among our engineers, and until the Society maps out a course of procedure for the guidance of the engineers of the future we need not expect any great improvement.

The desire to "save?" money, as so often seen in the conduct of railway "promotion" and construction work, especially if of the "bonus" description, is not conducive to good engineering results. On the contrary, it has to answer for many of the so-called shortcomings of engineers in the matter of general and final working plans and records.

VOL. XIV. 1900 PART II.

TRANSACTIONS

OF THE

# Canadian Society of Civil Engineers

OCTOBER TO DECEMBER, 1900.

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1900.

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OF

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The Society will not hold itself responsible for any statements or opinions which may be advanced in the following pages.

*"The papers shall be the property of the Society, and no publication of any papers or discussion shall be made except by the Society or with permission of the Council."—By-Law No. 47.*

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## INSTRUCTIONS FOR PREPARING PAPERS, ETC.

In writing papers, or discussions on papers, the use of the first person should be avoided.

They should be legibly written on foolscap paper, on one side only, with a margin on the left side.

Illustrations, when necessary, should be drawn on the dull side of tracing linen to as small a scale as is consistent with distinctness. They should not be more than 10 inches in height, and *in no case* should any one figure exceed this height. Black ink only should be used, and all lines, lettering, etc., must be clear and distinct.

When necessary to illustrate a paper for reading, diagrams must be furnished. These must be bold, distinct and clearly visible in detail for a distance of thirty feet.

Papers which have been read before other Societies, or have been published, cannot be read at meetings of this Society.

All communications must be forwarded to the Secretary of the Society, from whom any further information may be obtained.

The attention of Members is called to By-laws 46 and 47.

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1894.

LEX. PETE

IACDOUG

. ST. GEO

ALLIS.

BBOTT.

ARCHIBA

. BARNET

. BOVEY.

HANUTE.

. CUNING

ONKIN.

. DUGGAN

ALBRAITH

. GARDEN

HASKINS

. F. MACL

ETERS.

. RUTTAN

. VALLEE

KEEFER.

. S. GZOW

N KENNE

. HANNA

. MCLEO

V. BLACK

. MCNAB.

Thursday, 11th October.

DR. H. T. BOVEY, President, in the Chair.

Messrs. T. W. Lesage, L. A. Amos and R. Bickerdike, Jr., having been appointed scrutineers of the ballot for the election of Members, reported the following elected:—

MEMBER.

ASHLEY B. TOWER.

Transferred from the class of Associate Member to the class of Member:

A. J. STEVENS.

Transferred from the class of Associate Member to the class of Member:

E. J. BOSWELL, W. A. DUFF,
F. L. FELLOWES, J. HUTCHEON,
J. D. MACKERRAS.

STUDENTS.

T. V. ANDERSON, W. B. LINDSAY,
E. C. BAKER, E. H. LOWE,
H. L. BUCKE, W. R. MCCONKEY,
T. H. BYRNE, F. E. PATTERSON,
B. CALDWELL, E. H. PENSE,
W. HARTY, JR., J. A. RIDDELL,
G. B. JOHNSON, L. SHERWOOD,
W. B. YOUNG.

**Paper No. 153.**

# THE CARILLON CANAL, DAM AND SLIDE.

By Ernest Marceau, M. Can. Soc. C. E.

## I.

Before entering upon the description of the works named in the above heading, it may not be out of place to give a short history of what was done in former years towards improving the navigation of the second in importance of the Canadian rivers.

The Ottawa or Grand River, as any one who ever glanced at a map of the country knows, is the natural route between the head of ocean navigation at Montreal and the Canadian West, it being by long odds the safest and shortest. It was consequently the first used by European settlers in their trade intercourse with the western aboriginal tribes, as also by the discoverers, who, in the hope of reaching the shores of China, traveled in the direction of the setting sun.

It would therefore seem natural that this route should have been perfected before the alternative one of the St. Lawrence, but the better conditions of climate and the greater agricultural resources of the valley of the latter river soon brought the bulk of settlers towards it, and the rapid development of this part of the country, including the shores of the lower Great Lakes, promptly caused the Ottawa River to be only frequented by fur traders and later on by lumbermen, so much so that, up to the beginning of the present century, the Ottawa Valley above Bytown was still a wilderness with only a trading post here and there.

After the experiences of the American War in 1812, it became evident that some means of communication between the lower St. Lawrence and the upper part of the country, safer than those already in existence, must be provided, and the construction of what was termed the Military Canals was decided.

These comprised the Carillon, Chute-a-Blondeau, Grenville and Rideau Canals, the calibre of which was calculated to meet the requirements of the trade at the time, as well as those of the transportation of troops and military stores. The cost of their construction was borne by the Home Government, and they were man-

(1) The facts and figures relating to the Military Canals as well as to the original works on the Carillon Canal, Dam and Slide are compiled from the Canadian Blue books and other public documents.

aged and maintained by the Imperial authorities up to the year 1857, when they were transferred to the Government of Canada.

Work was inaugurated in 1819 on the Grenville Canal, which is situated on the north shore of the Ottawa alongside of the Long Sault Rapids. This canal, 5¾ miles in length, had seven locks, overcoming a rise of 48½ feet* and was located as shown on Plate No. I.

The original design for this, as well as the other military canals, contemplated locks corresponding in size to those of the Lachine Canal at the time, viz: 108x20 feet and, in 1828, the three upper ones had already been completed in accordance with this plan. During that year the Rideau Canal was commenced. The dimensions adopted for the locks being 134x33 feet, it was decided to make the locks remaining to be built on the Ottawa route of the same size, with the evident intention of increasing, at a later time, the dimensions of those already constructed.

The Chute-à-Blondeau Canal consisted of a mere cut, one-eighth of a mile in length, through a projecting rocky point on the north shore of the river, about 1⅜ miles below the foot of the Grenville Canal. The lock was 108 feet long, 32 feet 10 inches at the upper end and 36 feet 4 inches at the lower end, with 6 feet of water on the sills. No walls were built here except at both ends to receive the gates. As its name indicates, this lock overcame the Chute-à-Blondeau, the descent of which was 3 feet 9 inches.

About four miles below this point are found the Carillon Rapids, along which the river, in its natural condition, descended 9½ feet in a distance of a couple of miles.

The canal, 2⅛ miles in length, was located on the north shore, and, in order to avoid an extremely deep cutting, a summit level was provided, with a feeder from the North River, as shown on Plate II. The total lockage thus created amounted to 41 feet 6 inches, of which (in ascending the river) 25 feet 9 inches were upwards and 15 feet 9 inches downwards. Of the three locks built, the two at the lower end were combined ones.

The width of the locks on the Carillon and Chute-à-Blondeau canals, as well as in the lower reaches of the Grenville Canal, was about 33 feet, but the dimensions of the larger boats that could pass along from Montreal to Ottawa were governed by the smallest lock on the Grenville Canal, the width of which was only 20 feet, with a maximum available depth of 6 feet on the sills. On the other hand, the Rideau Canal, with its 47 locks 134x33 feet, could only admit vessels drawing 4 feet 6 inches.

With this limited accommodation, however, the traffic along the

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* By the construction of the Carillon dam the fall in the Long Sault Rapids was reduced to 45 feet 9 inches.

Montreal and Kingston route, as the line is called, amounted in 1858 to 222,241 tons.

About that time the Ottawa and French River navigation scheme was engaging the attention of the Government and, from 1858 to 1860, extensive surveys of the route were made by Messrs. Walter Shanly and T. C. Clarke.

Their most valuable reports, upon which is based the revived project of what is now called the Montreal, Ottawa and Georgian Bay Ship Canal, concluded to the feasibility of the scheme and put into strong light its numerous advantages, but, for reasons that are not to be detailed here, the project fell through.

The traffic on the military canals was, however, making immense strides; so much so that it more than doubled in the nine years between 1858 and 1867, this being in great part due to the rapid development of the lumbering operations along the Ottawa River.

Something had, therefore, to be done to help the ever increasing demand for transportation facilities, and consequently, in 1870, the Dominion Government caused surveys to be made for the enlargement of the Ottawa canals.

Construction work was commenced on the enlargement of the Grenville Canal in 1871, and on the Carillon Canal two years later. The latter was completed in 1882 and the former in 1887. The new locks, 200x45 feet, with 9 feet of water on the sills, can now admit vessels of 700 tons burden, and during the fiscal year 1893-94 there was carried through them a quantity of merchandise amounting to 562,010 tons, 80 per cent. of which consisted of timber, lumber, firewood and logs.

The above figures will give an idea of the importance of the Ottawa River route as a means of transportation for the country's products. We will now enter upon the true province of the present paper, viz: the construction of the works at Carillon.

## II.

As stated above, the Carillon Canal, as built by the Royal Engineers, had a summit level, with two combined ascending locks at the foot and a descending one at the head. The plans adopted after the survey made in 1870 by Mr. Andrew Bell, and which were practically identical with those submitted by Mr. T. C. Clarke in 1860, provided for the damming of the Ottawa and the building of a canal three-quarters of a mile in length, with two locks (the lower one only being a lift lock) overcoming the present total fall of from 16 to 17 feet, now accumulated at this point by the rolling dam thrown across the river. The only material difference between the two schemes is in the location of the canal, which has

been built on the north shore, while Mr. Clarke had intended it to be placed on the opposite side of the river. The dam was also devised so as to raise the reach above it to the extent of obliterating the fall at Chute-à-Blondeau and thus superseding the lock at that point.

By referring to Plate II, Fig 4, it will be seen that the new canal was formed by means of a heavy embankment built upon the river bed about 100 feet from the shore line. This embankment consists of clay and rock protected on the river side by cribwork along the toe of the slope. This toe crib is in turn protected against the action of the water in the river by rip-rap stone. On the canal side the bank is lined with a heavy rubble masonry wall laid in cement mortar and rendered watertight by a puddle wall at the back, 3 feet thick.

The lower entrance is protected by a pier 300 feet long and a similar pier, 800 feet in length, forms the upper entrance to the canal. This last entrance is extended some 600 feet further up by means of detached piers and booms.

From the head of the upper lock, upon which it abuts, the submerged dam stretches across the river in the shape of a circular arc measuring about 1,740 feet in length, its continuity, at least in the superstructure, being interrupted, about 400 feet from the south shore, by a timber slide, composed of three parallel piers each 615 feet in length.

At the site chosen for the dam, the Ottawa flowed through three distinct channels, divided by reefs with shallow water upon them: the South or "Canoe" channel, 100 feet wide; the Centre or "Sickle" channel, 400 feet, and the North channel, also about 100 feet in width.

The current at this point being very swift, some means had to be devised to create still water for laying the timber foundation of the dam, and, to this effect, the specification provided for the erection, immediately above the site of the permanent structure, of a skeleton bulkhead, also intended to support a bridge over which materials could be carried. This bulkhead, however, did not prove practicable, and was replaced by a temporary or false dam built of rough cribwork.

The foundation of the dam was brought to an elevation of 15 feet above the level of the mitre sills of the lower lock, and was built, in the shallow parts of the river, of timber sills scribed and bolted to the rock bottom, and in the channels, of continuous cribwork. On top of this foundation the superstructure was started and built up to 24 feet above datum, or of a height of 9 feet over the apron of the dam. It was at first intended to give the superstructure the shape shown in Fig. 5, Plate II, but for some reason, and, to the writer's mind not very wisely, this shape was altered to that shown on Fig. 6. By the change this superstructure was

made much lighter than was contemplated, and a direct fall of the waters from the crest to the apron created, which submitted the whole structure to dangerous vibrations. The additional length thus given the apron, a favourable feature in itself, did not, in the writer's opinion, compensate for the strain developed by the impact of the enormous mass of water and ice that, in time of flood, is carried over the dam.

A gauging of the river was made in 1872 by the Resident Engineer of the works, Mr. Andrew Bell, and here are the figures found by him:

At high water.....123,080 cubic feet per second
At low water..... 27,823 cubic feet per second

In times of flood, or for a period of three or four weeks, the sheet of water running over the dam is from 12 to 14 feet in depth. At low water it is only a couple of feet.

Considerable difficulty was experienced in laying the foundation cribs and sills, on account of the uneven condition of the river bed, and of the quantity of loose material (gravel and boulders) which was strewn over it. Another cause of delay was found in the shortness of the season of low water, not to mention the carrying away of some of the cribwork during the spring floods every year.

At the end of 1876, after three years work, the contractors had only succeeded in laying the foundations of the dam across the shallow parts of the river, or on a length of about 1,200 feet, and in partly building the slide piers.

The timber slide described in the specification was intended to be from 110 to 150 feet wide, with side piers extending 550 feet below and 65 feet above the dam. Operations on this portion of the works were commenced in 1874, the width of the slide being fixed at 120 feet.

In November, 1877, both slide piers were nearly completed, all the earth and rock excavation for the dam was done, the foundation timbers were laid across the entire shallow parts of the river, except for a distance of 100 feet on the north side, but no more than five cribs, covering a length of 120 feet, had been sunk in the channels.

At the canal the contractor's efforts had been somewhat more successful. On the above date the toe crib protecting the embankment was finished on a length of 3,166 feet, leaving some 300 feet still to be done; a large portion of the embankment was also completed, the bottom of the upper lock was laid and the masonry commenced; the prism of the canal between the locks was nearly finished, but the lower lock had not yet been touched.

The contractors, Messrs. R. P. Cooke & Co., who had all along

laboured under great difficulty, had given up their contract at the beginning of the year, and a general survey of the works already done had to be made for re-letting.

New contracts were awarded in June, 1879, one to the original contractors, Messrs. Cooke & Co. for the completion of the canal and locks, and the other to Messrs. F. B. McNamee & Co. for the dam and slide.

Mr. Andrew Bell had hitherto been in charge of the whole work, but, from the date of the new contracts, he only supervised the construction of the canal and locks, while Mr. H. Merrill was given charge of the dam and slide works.

The latter gentleman was given a pretty free hand, and very materially altered the original plans of the dam and slide. The shape of the superstructure was changed, as stated above, to a triangular one, the apex being lined with heavy boiler plate one-half inch thick to prevent its being worn away by the friction of the water, and the cribwork foundation in the channels, instead of being built continuous, was made of detached cribs in order to allow a freer passage to the water during construction. This was indeed carried so far that passageways were cut through the sills already put in place and through some of the cribs sunk in the channels. The dam was thus made of detached pillars, generally about 7 feet wide and standing from 5 to 12 feet apart; the open spaces were to be closed by heavy timber gates at completion. A continuous foundation was, however, provided in the deeper parts of the channels, but it was only brought up to a few feet above the bottom.

While securing the end desired and greatly facilitating operations, this mode of building could not but have a very injurious effect upon the strength and stability of the structure, not merely through weakening the foundation, but also on account of the soft nature of the rock upon which this foundation rests. This rock is shaly limestone, in beds only a few inches thick and very easily eroded. It was therefore to be foreseen that, unless the openings in the cribwork were very tightly closed, the rock alongside of the narrow pillars would gradually be washed away and a collapse fatally ensue.

It is not the intention here to follow the progress of the work day by day. Suffice it to say that in November, 1881, the dam, the slide and the canal were so far advanced as to admit of the closing down of the trap gates over the sluices. In May of the following year the new canal was opened to the traffic and the old one finally abandoned in August.

During the progress of the work the intended width of the slide had been reduced from 120 to 28 feet by the insertion of a third pier between the two original ones, it having been ascertained that the wide slide could not be satisfactorily worked.

This slide consists of the piers just mentioned, a timber breast wall, on the top of which stop-logs can be lowered from a building erected above, and a floating apron hinged on to the breast wall and intended to prevent rafts going through at low water from touching bottom. In the original scheme this apron was to be much longer and built solid from the bottom with an easy slope downwards.

While being closed down, some of the trap gates had been broken and carried away through the sluices. These openings were later on closed as well as possible by means of cribs, brush, stone and gravel, but not very effectively, as the sequel will show. This additional work occupied a few months more, and the dam seemed to be in perfect condition at the end of 1882.

The cost of the works was as follows:

| | |
|---|---|
| Dam and slide | $294,560 |
| Canal | 659,960 |
| | $954,520 |
| To which must be added damages awarded for flooding of farms above the dam, about | 17,000 |
| Total, about | $971,520 |

During the month of May, 1883, a crib was discovered floating down the river, and presenting the appearance of having been torn away from the dam foundation. The water being very high at the time, this could not be ascertained, but about the middle of June a large quantity of timber was seen rolling and plunging down the rapids below the dam, and a break became apparent in the surface line of the water above the crest. One or two of the foundation cribs had been carried away, as well as a large portion of the superstructure. The cause of the accident could be none other than the one pointed above. The rush of the water through some of the openings, left insufficiently closed, had rapidly undermined the rock; one crib had gone, and two others had soon followed, carrying with them some 170 feet of the superstructure, as was ascertained later on. The force developed by the water rushing through the gap was such that the rock bottom was excavated to a depth of 30 feet in the deepest part, upon an area of an oval shape 90 feet wide and over 150 feet in length, the sides being cut almost perpendicular. (Plate III.)

It was feared at first that the damage would not stop there and that the navigation of the canal would be impaired by the break, but

fortunately the level of the river above the dam did not seem to be materially affected.

Now arose the question of the repairs to be done. Nothing could be attempted until the river had subsided, but preparations were commenced without delay.

The first step to take was to enclose the break so as to create still water at the gap. This could only be effected by means of a row of cribs, sunk as close to each other as possible, above and parallel to the dam, and two wings built in the same manner. The building of this false dam was an undertaking of no small difficulty on account of the tremendous current to be contended with. A crib was first sunk 1,000 feet above the break, in comparatively still water, and from this a few others were launched by means of winches and steel cables.

The latter cribs were then used to launch, in the same manner, the cribs forming the enclosure as shown on Plate III. After long and persistent efforts, the gap was finally closed, and the small spaces unavoidably left between the various cribs filled with timber and plank. Soundings were then taken all over the place and the extent of the damage ascertained as given above, viz: 90 feet of the foundation had been carried away and the superstructure either destroyed or so damaged as to necessitate its complete rebuilding on a length of nearly 200 feet.

The deep hole excavated by the rush of the waters through the breach was first solidly filled with heavy cribwork extending from 18 to 25 feet below and about 20 feet above the dam, and a row of cribs 18 feet wide and 130 feet long were sunk close to the structure, both on the upstream and downstream sides.

The portions of the hole not occupied by this cribwork were filled up with boulders. The superstructure was then rebuilt and the apron widened and given a slight slope instead of being made horizontal as before.

Other portions of the dam were also examined at the same time and staunched by means of cribs and embankments formed of stone and gravel. The quantities of materials used, as well as the cost of both the temporary and permanent works, are given in the following statement:

## CARILLON DAM.

STATEMENT of the cost per cubic yard, of the Timber Work, Etc., Used in Repairing the Break of June, 1883.

| | Cubic Yards. | $ c |
|---|---|---|
| Cubic contents of temporary work..... | 51,243 | |
| Cost of temporary work..... | | 110,814.00 |
| Average cost per cubic yard of temporary work ..... $110,814 / 51,243 = $2.16 | | |
| Cubic contents of permanent work..... | 23,306 | |
| Cost of permanent work..... | | 33,120.80 |
| Average cost per cubic yard of permanent work ..... $33,120.80 / 23,306 = $1.42 | | |
| Total cubic contents of temporary and permanent work ..... | 74,549 | |
| Total cost of temporary and permanent work.... | | 143,934.80 |
| Average cost per cubic yard of temporary and permanent work..... $143,934.80 / 74,549 = $1.93 | | |

The repairing of the break was practically completed in October, 1884, or in about fifteen months, and the strengthening and staunching of other portions occupied another year. The amount expended on the latter work was $27,290, bringing the total cost of the repairs to $171,224.

The effect of this staunching was soon apparent, the level of the river rising over 12 inches at the foot of the Grenville Canal, a distance of 5½ miles, and the current at Chute-à-Blondeau, which was still very strong after the completion of the works in 1883, being reduced in such a measure that it is no longer an obstacle to the navigation of the river.

When the work was discontinued in 1886 it was known that some portions of the dam were not as strong and safe as they could be, although offering no immediate danger of collapse.

Four years later, in January, 1891, during a visit of inspection at Carillon, the writer's attention was attracted to the peculiar aspect of the water over the crest of the dam a short distance north of the

slide. A very marked break was noticeable at that point, and this was considered so serious that application was at once made for authority and funds to do the necessary repairs at the next low water.

Preparations were made during the summer, and the first coffer dam crib laid in October, 1891. The crest of the dam at that date was under some two feet of water, which had to be diverted.

The first portion of the dam that was unwatered was about 125 feet north of the slide, and it was then ascertained that the repairs had not been undertaken an hour too soon, for, as the various coffer dam cribs came to be loaded, the whole section was observed to sink down under their weight to a very appreciable extent, showing without a possible doubt, that that part of the structure was afloat and only kept in place by the resistance of the adjoining solid portions, which acted as abutments, the dam, as stated, being arc-shaped. Most of the anchor bolts driven through the apron into the rock bottom were found to have been drawn out, presumably by the vibrations to which the dam was submitted; some were sticking out almost straight above the apron covering, others had been curved down by the impact of ice or logs going over the dam during the spring floods.

The tamarack covering of the apron was first removed and the cribs underneath were found almost empty. Soundings taken immediately below revealed the fact that the rock there had been considerably scoured out. As to the covering itself, it showed a groove some two or three inches in depth, parallel with the crest of the dam, and caused by the action of water falling over it. The remedy to this state of affairs was evident. The empty cribs and spaces left open by the carrying away of the trap gates had to be refilled, and, to render this possible, a row of cribwork was built close to the down stream side of the apron, so as to keep the filling in place. The apron was in that way lengthened up to 25 feet in shallow places and to 50 or 55 feet in the channels. Moreover, in order to increase the weight and stability of the structure and to prevent the dangerous vibrations caused by the impact of water falling over the apron, the shape of the dam was changed, by raising the apron and giving it a slope as shown on Plate I, Fig. 7.

A heavy covering of rock elm, hewn or sawn on three sides, at least 10 inches thick, was laid, the round side up, over the crib forming the apron, these cribs having previously been very carefully filled with boulders, of which there is an abundance in the neighbouring fields. Particular precautions were taken in laying this covering, each stick being bolted with 1-inch square bolts, first to the framework of the cribs and again transversely with the stick alongside of it, so as to form one solid platform, no portion of which can be removed without tearing up a large area of the whole.

The repairs were carried on from October, 1891, to the end of March, 1894, with interruptions in the fall and at the breaking of the ice in the spring, the work being done in sections of from 120 to 220 feet in length.

After the main body of the dam had been made safe in the manner described, attention was directed to the section between the slide and the south shore. Here it was discovered that the pressure against the southernmost slide pier had been such as to throw it out of alignment, so much so that it was in danger of collapsing. When reducing the width of the slide by building a third pier between the two first ones erected, it had been thought that the weight of the southernmost pier would be sufficient to hold the dam in place and the breast wall of the slide was not carried across the space between the two southern slide piers. The result of this miscalculation was as stated above. It now became necessary to fill the gap thus left, and a strong cross pier was built in the position shown on Plate IV, the space between it and the stop logs which formerly closed the opening, being filled with boulders. The outer south pier had then to be torn down and rebuilt from the top to the level of the water for a length of 250 feet or so.

During the winter of 1893 soundings were taken above the dam, and wherever the bottom or the embankment was found to be more than 2 feet below the lower edge of the superstructure, boulders were dumped until the holes were filled up.

A curious experience may be related here. Large quantities of frazil drifted down and accumulated against the up-stream side of the dam at places where the overflow was stopped by the coffer dams. At one point a trench some 4 feet wide was sawn through the ice covering and boulders dumped on top of the blocks thus detached, but although piled up 5 or 6 feet in height, several days passed before they sunk out of view. All this time an iron bar could easily be driven through the spongy mass below, the depth of which was from 10 to 12 feet.

The total cost of the Carillon Works at the present day is as follows:

| | |
|---|---|
| From 1873 to 1882 | $971,520 |
| Repairs, 1883 to 1885 | 171,225 |
| Repairs, 1891 to 1894 | 54,300 |
| | $1,197,045 |

## III.

A mere description of engineering works, although interesting in itself, would remain very incomplete if it did not go a step further. What is of greater interest is an account of the difficulties

encountered in their execution and of the methods adopted in overcoming these difficulties. Every one of us knows how to build cribwork or masonry under ordinary conditions; but every one of us wants to learn by what means, conditions being out of the ordinary, cribwork or masonry can be built at the minimum cost and with the least trouble.

To give the members of the Society the benefit of one's experience in that direction, the writer believes, should be the ultimate object of all papers published in our transactions. It is, therefore, proposed here to briefly describe the plant used in the repairs executed at the Carillon Dam between 1891 and 1894, of which the writer had immediate charge, and show by what methods the large quantity of materials employed was handled.

The Ottawa River, above the dam, flows with a velocity of from 6 to 10 miles an hour, according to distance, and the handling of timber and other materials there was of itself a difficult operation, especially as it had to be done immediately above a vertical fall of 15 feet, followed by half a mile of raging rapids.

The crest of the dam, as previously stated, is immersed a couple of feet at the lowest stage of the river, and a body of water 12 to 14 feet in thickness rolls over it in times of flood. It is needless to say that no work could be done during the periods of high water, and operations had to be stopped at the beginning of each spring, to be resumed about June, and suspended again in November until the ice formed.

The unwatering of the dam, or at least of its portion below the crest, was the first problem to solve. This was done in sections of from 125 to 220 feet, commencing at the north side of the slide.

The north slide pier and the guide piers above it, with the booms stretched between them formed a base of operations, and greatly facilitated the work on the first section. The object in view, besides the unwatering of the dam, was the creating above it of a still water basin, or at least of an enclosed pool where timber and scows loaded with stone could be brought without being in danger of being carried away by the current. One side of the closed pool was constituted by the slide piers and booms, as stated above; similar booms, anchored to piers built for the purpose, were stretched on the opposite or north side of this section, and the front of it consisted of the coffer dam proper, which will now be described. (See Plate IV.)

The superstructure of the dam was used as a part of this coffer dam. It was then in the shape of a right-angled triangle, the hypothenuse of which rested on top of the foundation cribs, and the longest side of the right angle facing the current. It being impossible to build the coffer dam in place, cribs, the bottom of which

was shaped to conform with the pitch of the up-stream side of the superstructure, were framed and put together on shore. Platforms were provided at each corner of them near the top, and, as each crib was launched and brought to deep water, stone was laid by hand on those platforms, so as to merely balance it without sinking it too much. A steam tug then towed it as close to the dam as the current would safely permit, and by means of hawsers, paid out from winches on the tug, and on piers previously sunk in convenient positions, it was lowered very slowly and carefully till its bottom rested on the up-stream side of the dam. While it was held in this position by its weight and by the ropes, men approached it in a rowboat and corrected its position so as to bring its down-stream face directly over the crest; it was then loaded with stone and the top platforms removed. The same operations were repeated until as many of them were placed as were required to cover the whole section to be unwatered. However, in order to reduce the expenditure, these cribs were laid from 20 to 30 feet apart and the space between them closed by stop logs. When in position the cribs were far from being watertight, but their down-stream face was framed so as to permit of the insertion of close sheet piling, as shown on Plate IV, Fig. 2. Leaks were then caulked and the apron was effectually unwatered to within a few feet of both ends of the section. Whole bands of timber were floated down against the coffer dam and transferred to the apron below by means of a sluice cut into the stop logs. The cribs for the lengthening of the apron were built in place, that is, close to the down-stream side of the old apron, firmly anchored to the rock bottom and filled with boulders. This having been done, the covering of the old apron was removed and put aside to be used over again. The old apron cribs, which were more or less empty, and the open spaces left in the foundation of the dam were then carefully filled with boulders and a continuous superstructure built on top of the old and new apron cribs, binding them together, and sloping down stream so that when the covering was laid its lower end stood about 4 feet above the level of the old apron, its upper end butting against the original down-stream face of the superstructure, about 12 inches below the crest. (See Plate IV.) When the first section had thus been completed, the outer coffer dam crib was left in place and the other ones brought round and used on the following section in the same manner.

The above gives a fair idea of summer operations. About the end of November work had to be suspended and preparations commenced for the winter, and, as soon as the ice bridge was formed, work was resumed and carried on until the end of March or the beginning of April, when all the stop logs were removed and the coffer dam cribs overloaded, so as to resist the tremendous pressure of water and ice during the spring freshet.

About the end of December the coffer dam was backed up by a sheet of ice which gradually extended up-stream and gained in towards the shores, only leaving open water for a short distance above the portions of the dam which were not being worked upon. However, during the mild winter of 1891-92, the middle of January was reached before the bridge had formed from the north shore, and it then became apparent that, unless some means of helping the ice to take were found, the whole season would be lost. The open channel, which was about 300 feet wide, was running full of frazil and anchor ice, and the problem was to stop these at one point at least.

At the outer edge of the channel there stood out of the water a pier which had been used the previous summer to launch coffer dam cribs. Between this pier and the shore ice strong manilla ropes, 3 inches in diameter, were stretched by means of winches and brought to within a few inches below the water surface at the centre; small cedar trees were tied to the ropes and allowed to drag in the water. Soon some of the floating ice was arrested, but the process being very slow, men were sent up to Carillon or Dewar's Island, a short distance up-stream, and sawed out large pieces of shore ice, which were floated down towards the barrier. Finally, after seven or eight days, a nucleus was formed around which the water consolidated in one night, closing the channel. As soon as this ice was strong enough to bear the weight of a man, brush was laid over it and abundantly sprinkled with water; a plank road was laid at the most dangerous places, and work resumed about the end of January.

A rather unusual method was adopted for filling cribs in places where the apron was very wide. The top of the coffer dam stood about 15 feet above the level of the apron, and where it was possible to do so, the stone was thrown from sleighs in winter and from scows in summer, down plank chutes, directly into the cribs. In two or three places the old apron was from 25 to 30 feet in length, and chutes long enough to reach the new crib below this would have been impracticable. However, in order to avoid a second handling, the chutes were used to carry the stone down near the lower end of the old apron, where a pile was formed, and the sluices in the stop logs were opened, when a powerful stream of water, under a head of about 12 feet, was directed by means of the same chutes, against the piles of stones which were thus thrown into the cribs. This stream of water performed the work of at least twenty men.

As to the cost of the repairs, it may be stated that from October 1891, to April 1894, some 23,000 cubic yards of permanent cribwork were built at an actual outlay of $54,200, to which must be added the value of a large quantity of timber left over from the work done between 1883 and 1885, and which was used in the false works. Tak-

ing everything into account, the average cost of the cribwork was about $2 per cubic yard, which, the writer thinks, is a somewhat remarkable result, considering the conditions under which it was attained. It must be said, however, that timber can be purchased at very low figures at Carillon, this point being on the way of rafts coming down to Montreal. The average cost of the timber used, comprising rock elm, tamarack, spruce and hemlock, did not exceed 10 cents per cubic foot. Boulders, of which there is an abundance in the vicinity, were delivered on scows in summer at 30 cents, and in the cribs in winter at 40 cents per cubic yard.

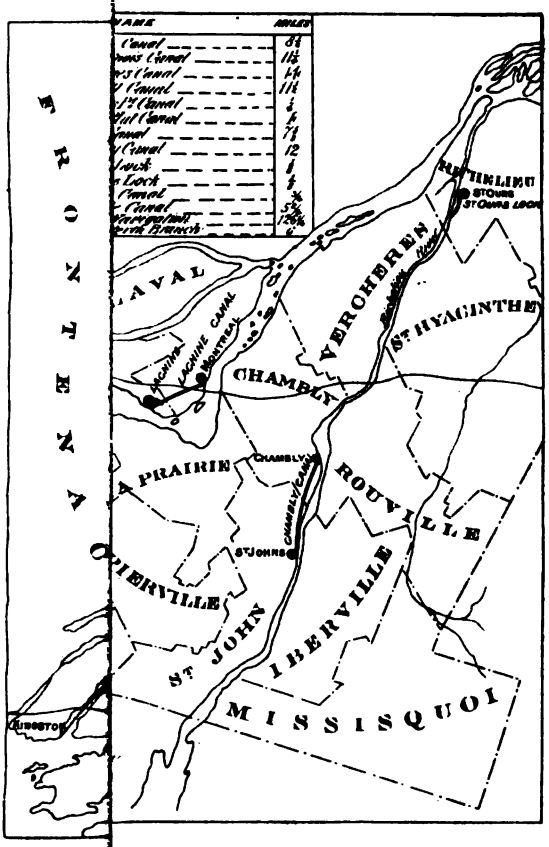
A point which it is thought deserves to be specially mentioned is the fact that in the thirty months of actual work, during which these repairs were carried on, and with a force of from 40 to 120 men, not a life was lost and not a man was seriously injured, notwithstanding the extremely dangerous conditions under which the work was performed. A greater compliment could not be paid to the person in immediate charge, Mr. D. Gaherty, an old experienced river and timber man, to whom a large part of the credit for the general results above given is due.

Before concluding this rather lengthy paper, the writer would remind the members of the Society who take an interest in the development of water power, of the existence, at Carillon, of an immense amount of energy which has been going to waste for over fifteen years.

The power was there before, but it was only on the completion of the dam, in 1883, that it was accumulated at one point. The discharge of the Ottawa River here, as measured by Mr. Andrew Bell, in 1872, is 123,080 cubic feet per second at high water, and 27,823 feet at low water. The minimum head under which it falls is about 12 feet, and 2 or 3 feet more could be gained at very reasonable cost.

It may be stated that some capitalists seem to have lately valued the possibilities of this magnificent water power, and have secured from the Dominion Government a lease of more than half of it. It is to hoped that they will lose no time in utilizing it. This is an ideal site for a pulp mill, it being in close proximity to an almost inexhaustible supply of pulp wood and possessing unsurpassed facilities for shipment, both by water and by rail.

ANSACTIONS CAN. SOC. C. E.
VOL.XIV. PLATE.I.



Thursday, 25th October.

DR. H. T. BOVEY, President, in the Chair.

The following donations to the Library were reported:—

"Locomotive Engineering and the Mechanism of Railways," from Mr. H. W. D. Armstrong, M.Can.Soc.C.E.

"Hydraulic Power Engineering," from Mr. J. W. Harris, A.M.Can Soc.C.E.

It was unanimously resolved to recommend to Council that all dues be remitted to members who have been absent from the country as volunteers to the South African war.

*Paper No. 153.*

## ROAD CONSTRUCTION AND SYSTEM OF CONTROL.

By A. W. CAMPBELL, M.CAN.SOC.C.E.

The volume of literature respecting road construction and maintenance has assumed very considerable dimensions, and is rapidly increasing. This branch of engineering cannot be said to present problems of very serious difficulty, and the underlying principles are well understood. They have been very largely applied in the construction of Canada's magnificent system of railways, which are, in effect, but the trunk highways provided with tracks of steel, and steam in place of animal traction. A study of the map of the early common roads in Ontario, prior to railway construction, will indicate the remarkable degree to which railways have followed the lines of the first main roads.

The most famous roads the world has known were built 2,000 years ago, by the Romans, many of which are still in use. So durably have they been built that, in Spain, and other European countries where they exist, they are believed by the peasantry to be of supernatural origin. They commonly range in width from 40 to 120 feet. The method of construction was to first excavate to hardpan. This was filled with layers of sand, stone and concrete up to the required elevation; and this was maintained in a direct line and with easy gradients, at an enormous expenditure of labor on cuts and fills. While the permanency of these roads may receive our admiration, yet they can scarcely command respect, built as they are at such a prodigal waste of energy.

What a road must be, to be a good road, should not be misunderstood. A good road is one best answering, in a broad sense, the

requirements of economic fitness. Money spent on road construction is an investment from which adequate returns are to be expected. A road, the cost of which is excessively great in proportion to the use made of it, however smooth, hard, easily traveled and durable, measured by the foregoing rule, may be far from being a good road. On the other hand, a common dirt road may be a good road if it is made and maintained so as to properly meet the needs of traffic. In Canada, therefore, we do not need English, French, nor Roman roads, but we want Canadian roads, each mile adapted to local requirements and conditions.

It is impossible to make a fixed model for a road to meet all conditions. There is a tendency on the part of many very good authorities on road-making to lay down hard and fast rules based on their own experience. This is a very great mistake, for in no other branch of engineering, perhaps, is there so much opportunity for the exercise of individual judgment. There are such infinite gradations in climate, in the character and extent of travel to be served, the nature of the soil over which the road is to pass, the opportunities and need for drainage, the quality of metal obtainable, that there is but one way to skillfully meet these conditions. It is to thoroughly understand what a road consists of, and the physicial properties of the materials which are to be employed in reaching this end.

A road consists of a strong foundation capable of supporting any load or weight which may pass over it. This foundation must have a surface covering which will resist the wear of the feet of horses and action of wheels, remaining smooth and hard; a covering which will also prevent water from passing readily through to the foundation. In the road, therefore, we have a structure consisting of a foundation, floor and roof.

The practice of the ancient Romans, in providing a foundation, was, as we have seen, to excavate to hardpan, the place of the excavated earth being taken by layers of hard material. But, a little more than a century ago, first in Switzerland and France, then in England, this plan was greatly changed, by the discovery that the natural sub-soil could, in the great majority of cases, be made sufficiently strong to support traffic, by the simple expedient of keeping it dry. This is the fundamental principle of what is generally known as the "Macadam" road.

The name is commonly applied to any road surfaced with broken stone, and in this respect is a very unfortunate misnomer. It is the neglect to provide a dry subsoil that is the greatest cause of the unfortunate condition of roads throughout Canada to-day. The roads of to-day are, in effect, but a repetition of the English roads as they existed before the time of Macadam; they are the roads which the system of Macadam displaced.

A road bed in which sub-drainage is not sufficiently provided, is the antithesis of a Macadam road. The importance of keeping the subsoil dry cannot be too thoroughly impressed. Clay in thick beds, when dry, will support from four to six tons per square foot of surface, according to the quality of the clay. If only moderately dry it will support only from two to four tons per square foot of surface. If the clay is wet and soft, it will yield to almost any load. Gravel, if well compacted, forms a much stronger roadbed, is less yielding to the action of moisture, and for this reason, even for a thin surface coating, strengthens the road somewhat. But the real strength of the road must lie in the subsoil. Vegetable moulds and alluvial soils are weak, having a sustaining power of only one-half to one ton per square foot, and for this reason it is well to remove such soils, securing, if possible, a gravel, clay or sand foundation.

A dry subsoil becomes, too, of paramount necessity in a cold and humid climate, such as prevails throughout Canada for a considerable portion of the year. The injury done to roads by frost is caused entirely by the presence of water. Water expands on freezing, and the more there is under a road, and above the frost line, the greater is the injury. In freezing, the particles of soil in immediate contact with the water are first compacted. When room for expansion ceases within the body of the soil itself, owing to its saturated condition, the surface is upheaved. When thawing takes place, the subsoil will be found honeycombed, ready to settle and sink beneath traffic. It is, therefore, of the utmost importance that the soil should be relieved of all water of saturation as quickly as possible by underdrainage. The impassable condition of the roads during the spring, often axle-deep with mud, is to be attributed very largely to a wet subsoil, which has been honeycombed in this manner.

The making of a strong foundation thus resolves itself into a question of underdrainage, and the means whereby proper underdrainage is obtained must be adapted to the manner in which water finds its way under the road, and the nature of the soil. A soil retains in its texture, by capillary attraction, a certain amount of water. In the case of a plastic clay, which will absorb nearly one-half its weight and bulk of water, the water retained in this way may be the cause of injury. In the case of gravelly, sandy, or other porous soil, it is necessary to remove only the water held by hydrostatic pressure in the foundation of the road. The effect of this is, that, with a clay subsoil, underdrains are nearly always beneficial in securing a strong foundation, and are necessary for traffic of even a moderate degree. With porous soils, on the other hand, the necessity and means of drainage will depend upon the height to which water rises in the foundation, and the direction

from which it comes. When a strong foundation is needed, these underdrains should be three or four feet below the surface of the subsoil. Their location with respect to the road should be varied with circumstances.

The most effective type of drainage employed is a system in which there is a tile drain on each side of the roadway, underneath the open gutters, with V-shaped drains at intervals, from the centre of the roadbed to the side drains. From this the scale descends to drains at the sides of the road only; then a drain at one side only, or in the centre of the road; then only an occasional drain at springy or damp points.

It is of advantage to understand the manner in which the underdrains act in different cases. With porous soils, in which the water rises under hydrostatic pressure, the water enters the tile from below; just as water rising in a vessel finds an outlet in the side or flows over the top, so the underdrains supply the necessary outlet for this excess moisture at a proper depth from the surface; it "lowers the water line."

With clay the process is different. Absorbing and holding, as it does, like a sponge, a large quantity of water, drains are less effective, but none the less necessary. The cracks and fissures which appear throughout the surface of a sun-baked clay soil during the summer drought afford a clue to the action of underdrains upon such a soil. As the clay yields up its moisture, it shrinks, is torn apart. These fissures, commencing at the drain, spread and branch in different directions, and each fissure thus becomes a new drain leading water to the tile. This process goes on, the fissures become filled with sand, vegetable and other porous matter, so that they assume a degree of permanency, and, in clay soils underdrainage is more effective after several years than at first.

The metal covering placed over the earth foundation should perform two services. It should be waterproof, so as to prevent water from readily passing through and softening the subsoil; and it should form a smooth, unyielding and durable surface covering to facilitate traffic and resist wear. The effectiveness of the stone covering as a water protection is increased by crowning the roadway so as to throw the water to side drains or gutters, which, in turn, carry the water from the road.

The varieties of stone which bind most readily, as a rule, will form the better protection from water. But, in order to be a good protection, the road surface must be smooth, without ruts or hollows, otherwise the flow of water to the gutters will be impeded. The water will then rest in the wheel tracks or depressions, will percolate through the surface, and traffic will immediately create injury. The stone, therefore, which binds readily, may not be a good covering, unless it is tough enough to resist wear and main-

tain a smooth surface. While the name stands for much, it is not a guarantee, by any means, of the usefulness of stone for road purposes, since each variety may vary much, even in the same quarry. The French Department of Public Highways has adopted a scale from 0 to 20, showing the quality of different kinds of stone. This list indicates:

| | | | |
|---|---|---|---|
| Trap | varying | from | 16 to 20 |
| Quartzite | " | " | 11 to 19 |
| Quartz | " | " | 10 to 18 |
| Limestone | " | " | 5 to 17 |
| Gneiss | " | " | 5 to 17 |
| Granite | " | " | 3 to 16 |

These rocks are among the most common in Canada and the list serves to show the wide degree to which their usefulness for roadmaking may vary. Limestone is most frequently criticized, but French practice shows that it may rank among the best; while granite, so frequently extolled, may be very poor indeed. Limestone, while it is very apt to be soft, is one of the best binding stones, and for this reason a combination of moderate toughness with good binding qualities may result in an excellent stone for a Macadam road. Trap is universally regarded as very reliable, since it combines both toughness with fair cementing properties. The difficulty with granite, on the other hand, is that, while very hard, it wears into a coarse, sandy material, which refuses to bind.

The "binding" of a road covering is a matter very frequently discussed. Some road engineers advocate the use of a foreign binding material mixed with the stone, such as sand, loam, or even clay. It is generally conceded by the best authorities, however, that the less foreign material incorporated with the stone the better. The best roads to-day are bonded by intermixing with the broken stone, the chips and dust created in crushing, and by thorough rolling. The dust of stone suitable for roadmaking will form a cement which will unite and re-unite after being broken, and added to this is the mechanical clasp which the stones assume after sufficient rolling.

The use of sand as a binder, while it will lessen the amount of rolling required to produce a hard surface, tends strongly to injure the durability of the roads. When these foreign binders are used, the road is more easily permeated by water, and in wet weather, such as we have to guard against stringently in a Canadian climate, the bond is less perfect, and the road is more readily rutted, the presence of sand having prevented as strong a mechanical interlocking of stones.

The quantity of stone required to form the surface covering will vary with the strength of foundation obtainable and the nature

of the traffic to which the road will be subjected. If, because of moisture or the nature of the soil, the foundation is lacking in strength, or if the traffic is to be very heavy, the Telford system may be employed, whereby a layer of large stones is first placed by hand over the road, and this is then covered with broken stone. When a Telford foundation is laid, the principal matter to observe is to place the stones on edge, either at a uniform height, or graded from the centre to the side, the highest at the centre. They should be given an even bearing, projecting tops should be chipped off, and spalls wedged into the interstices. If the stones are laid flat, they do not retain as firm a bearing, and are apt to rock, when uneven settlement will occur.

The depth of broken stone to be placed over a Telford foundation need rarely exceed six inches in thickness, the foundation itself having a thickness of eight or nine inches. Without the Telford foundation, the thickness of stone may range from six to twelve inches, according to circumstances. Less than six inches has been used successfully, but in such cases the foundation must be of unusual strength, such as a very gravelly soil would produce. When there is reason for the depth of the stone reaching or exceeding twelve inches, it generally indicates the necessity for a Telford foundation.

The broken stone should be placed on the road in layers of from 2 to 4 inches in thickness, according to the size of the stone, and each layer thoroughly rolled before the next is placed above it. The stone, too, should be first graded according to size, and the coarser placed in the bottom. When fine and large stones are intermixed, the surface will wear less uniformly, the smaller stones wearing more rapidly than the larger; and large stones, too, obtain a less firm bearing and are apt to work loose; so that, unless the finer stones are placed on top, the surface is apt to become rough, particularly in dry weather.

The preliminary earthwork, in providing for the reception of the broken stone, will vary greatly in extent. A cheap form of country road can be built by first crowning the earth subsoil by the use of a grading machine; then reversing the process, and, instead of bringing the earth in, cutting off the crown, turning it outward to form shoulders for confining the narrow bed of metal. From this method, the extent of earthwork will increase until the widest and most elaborate form of Macadam road is reached, necessitating the removal of much earth to receive the stone.

The location of roads is a matter with which, unfortunately, the Canadian engineer has not much to do. They follow, as a rule, the lines of original survey, and any effort to better their location is apt to be met with much opposition, interfering, as it would, with boundaries of private lands. The location and grade are mat-

ters which should be considered together, and should be the result of a skillful compromise. They are not matters of at all so much importance with common roads as with railway construction, but should not be neglected. The better the road, and the larger the loads which are to be hauled over it, the more important grade and location become.

The grade for ordinary roads may follow the general surface of the country. So far from being a benefit, a long and constant grade is more fatiguing for horses than is a gently rolling surface. The highest permissible grade on a Macadam road is 10.5 per cent.; an effort is generally made to prevent it exceeding 7 per cent.; while 3 per cent. is not objectionable. There should be at least sufficient fall to provide proper drainage.

The height of crown may vary with the quality of the road. An ordinary gravel road, when finished, may have a slope of one inch to the foot from centre to side. But a properly built broken stone road, when finished, should have a slope of about two inches to the yard, which, after a little wear and settlement, will be reduced to about half that amount of crown, or one inch to the yard. On hills the crown should be sufficient to prevent the water flowing down the road in the wheel tracks, which, if permitted, will quickly deepen them to ruts.

The use of machinery in road making is taking a prominent place, as in other branches of construction. Wheeled scrapers, road graders, rock crushers with screen attachment, steam and horse rollers are the more common. The grader is most useful on country roads of cheap construction, such as must serve a great proportion of Canada for a long time to come. The crusher is of less universal value, but is gaining popularity wherever there is rock to be broken or coarse gravel or boulders to be treated.

A heavy roller is becoming more and more regarded as indispensable in building a Macadam road, and the best results cannot be obtained without one. Unless a roller is used, the stone must be spread loosely on the road, and left for traffic to consolidate. This is a slow process, causing much inconvenience to travel, and during which the earth and stone become intermixed. Earth mixed with stone, as previously pointed out, prevents the strong mechanical bond which clean metal will assume when the stones are wedged one against another. The particles composing this earth, in wet weather, too, move more readily around one another, the road surface is softened, and is more readily broken up. By the use of a roller, the earth subsoil should first be thoroughly compacted. Each layer of stone placed upon it should also be well consolidated. In this way a smooth, durable, waterproof coating can be laid over a firm foundation.

There are different kinds and classes of rollers. The horse roller,

weighing six or eight tons, will do fairly well if a steam roller cannot be afforded, but the horse roller is not sufficiently heavy, and has to be used much longer on a given section than a steam roller to produce the best results. The feet of horses, in exerting sufficient strength to move the roller, sink into and disturb the road metal, and thereby injure the shape and quality of the roadway.

There is danger, on the other hand, of having a steam roller which is too heavy. A very heavy roller will sometimes sink into light or loose soil, force it ahead, and create a mound over which it cannot pass; this, however, may sometimes be overcome by spreading over the surface of the soil being rolled a thin coating of gravel. The same result will sometimes occur with an excessively heavy roller on a layer of loose stone. The heavy roller is more liable, too, to injure underground pipes, catch basins, culverts, bridges, or disturb sidewalks.

For these reasons a roller exceeding ten or twelve tons in weight in some localities, where the soil is of a loose or sandy nature, is frequently not desirable. In districts where the natural soil is gravelly or of stiff clay, a heavier roller may be operated successfully, but some municipalities have found it necessary to use a light horse roller in consolidating the subsoil and first layers of stone. Nor, if the stone used in the construction of Macadam roads is of a soft nature, is a heavy roller, say of twenty tons, desirable even in the finishing courses, as the crushing effect has been found in some cases to crumble and pulverize the stone rather than merely consolidate it. For new work, in which the dirt foundation must be rolled, a weight of twelve tons is generally the most serviceable; but for picking up an old roadway and reconsolidating it or for finishing a new work, fifteen tons is better. Where a municipality is to own only one roller it will be advisable to consider very carefully the work to be done before purchasing a roller of over twelve tons weight.

By proper attention and repairs the life of a broken stone pavement can be made continuous. The surface can be frequently rolled, improving it greatly. It can be scraped and swept, as are other pavements. When it begins to lose shape, the surface can be loosened up by means of teeth attached to the roller, a light coating of new metal applied, and then rolled down as well as when new. It is by such means as these that broken stone roadways can be made more economical and satisfactory than any other for roads and streets generally. This ease of renewal and repairs is a property peculiar to Macadam, which renders it most suitable for general purposes. It forms a permanent basis, and its perpetuation is merely a matter of repairs.

That the country roads throughout Canada are in an unfortunate condition is now very generally recognized. This, a careful study

will disclose, is not wholly due to the fact that the expenditure upon them is small. It is rather due, in many districts at least, to the fact that the expenditure now made is mismanaged and misapplied, owing to the lack of method, and an ignorance of the principles of roadmaking on the part of those in charge of the expenditure.

In Ontario, money and statute labor to a combined value of $4,000,000 are annually placed on the roads and streets throughout the Province. This is obviously a very considerable amount of energy; it indicates the importance of the highway as a public work, and demands a proper system of direction. That such a proper system has been utterly wanting requires no assertion other than the proof given by the roads themselves. Not too soon can the prevailing opinion be cleared away that any and every farmer is a competent road engineer. There is imperative need of an early introduction of business and scientific principles in the management of Canadian roads.

The public may, no doubt, readily see that for every elaborate road system the services of an engineer are necessary; but they have yet to learn that, wherever the possible expenditure is small, there is all the more need for conserving to the utmost the energy available, and that for this reason skill and experience should be employed by every municipality.

In contradistinction to the present statute labor system under which the roads of Canada are, for the major part, constructed, and maintained, it is essential that there shall be a very radical change, and that responsibility and control shall be centralized and defined. This should rest, primarily, with the Provincial Governments, and a subdivision subsequently made between county and townships, the former controlling the main roads within its area. This control and subdivision of power can only be exercised by each party thereto bearing a share in the cost. The principle of state aid is one which, for this, as for other reasons, is essential, creating a means whereby the people of the cities and towns may assist in the work of country road building.

The principle that all citizens shall contribute to the cost of roads is already being carried into effect on this continent, and exists in all European countries. There does not appear to be any reason why the farmer and rural population should pay the entire cost of road building any more than they should meet the entire cost of railway and canal construction. A system of good country roads is an expensive public work, in every way necessary for the development of the country, and, so long as the farmer bears the entire burden, it is manifest that the desired end, good roads, will be difficult, if not impossible, to reach. In any event, the attempt to do so comprises an injustice.

The towns are asking for good country roads. They are beginning to realize how important for them it is to have free and uninterrupted communication with the surrounding country districts at all seasons of the year, and would no doubt be willing to pay a fair proportion of the cost, in order that road improvement would progress more rapidly. This very interest which is being displayed is the strongest evidence as to the mutual right of townsman and farmer to pay for the construction of a country road system.

In Massachusetts, on petition of a county, the Road Commission may, with the assent of the Legislature, adopt any road within a county as a state highway. Except that the grading and bridging are done by the county, the work thereafter, both the construction and maintenance, is under the authority of the State Commission. Also on petition of two or more cities and towns, a road between them may be made a state highway. The State Commission is composed of three Commissioners, who compile statistics, make investigations, advise regarding road matters. One-fourth of the cost of construction is paid by the county, the remaining three-fourths being paid by the state. In 1894 the state spent $300,000 in this way; in 1895, $400,000, and in 1896, $600,000. It is intended that ultimately about one-tenth of the entire road mileage will be built as state highways.

In Michigan, upon a majority vote of the ratepayers in any county, a county system may be adopted. A board of commissioners, five in number, are elected by the people, to lay out and construct certain of the leading roads, to be paid for and thereafter maintained by a county rate.

Last year (1898) an act was passed by the New York State Legislature, providing that, on the petition of a county council, certain roads may be adopted as state roads. The petition is first presented to the State Engineer. If he approves of the section of road thus sought to be improved, he prepares plans, specifications and estimates. These are presented to the Legislature, and, if approved by that body, fifty per cent. of the cost of construction is paid by the State.

The New Jersey highway law provides that, on the petition of the owners of two-thirds of the land bordering on any road, the State Commissioner of Public Roads will cause the road to be improved in accordance with plans and specifications prepared by him, subject to the approval of the Legislature. The owners of the land affected by the improvements pay one-tenth of the cost, the county pays six-tenths, and the State three-tenths.

Connecticut has introduced a plan of highway improvement providing for the appointment of three State Commissioners. When a township votes in favor of constructing a road, under the provisions of the State Highway Act, specifications are prepared and

submitted to the State Commissioners. If the Commission approves, the township council lets contracts for the work, to be performed under the supervision of the State Commissioners. One-third of the cost is paid by the state, one-third by the county, and one-third by the township. The expenditure by the state in this way is limited to $75,000 annually.

The State of Rhode Island has appointed a Commissioner of Highways. When a council represents to the Commissioner the need for improving a certain road, an examination is made by him. If he considers the work necessary, he prepares plans, specifications and estimates, and reports to the municipalities affected; also to the State Legislature, as to the proportion in which the expense should be met by the state and the municipalities benefited. If the tate Legislature approves, the work is performed by contract.

Vermont and California also contribute largely in the form of state aid, while Indiana, Kentucky and others contribute to a less degree. In all these systems, safeguards are placed to prevent the expenditure exceeding for any state or locality, certain reasonable limits, according to the requirements and ability to meet the payments. In most of the states the tax is so levied that the towns and cities pay the greater portion of the cost of state road construction; for example, in the State of New York it is estimated that the people outside the towns and cities pay only 10 per cent. of the cost.

In England the main roads are under the control of the County Council, and are managed by the county engineer. The maintenance of the roads of lesser importance rests with the district councils. A central authority, the Local Government Board, is entrusted with a general superintendence for the benefit of the nation at large. For a number of years a grant was made from the Imperial Exchequer towards main highways, but since the roads have passed the stage of a first thorough construction, this has been discontinued, it having been found that the counties since that time have been fully able to meet the necessary expense.

French roads are national, departmental and communal; corresponding largely to state, county and township roads. The national roads radiate from Paris, extending to all the important cities and departments, and are under a special engineering department (Department of Bridges and Roads), attached to the national government. The second and third classes, departmental and provincial roads, are, in a general way, under local authorities, but departmental roads are usually entrusted to the care of the National Corps of Engineers.

Germany has a magnificent system of turnpikes, built and maintained by the national government. They are under the general management of a state road commissioner, while he is assisted by

an extensive staff of road directors and inspectors. Other roads are known as "country roads," and are built and maintained by the several parishes through which they pass.

The highways of Austria are classified as state or imperial roads, provincial roads; district roads, and community roads, according to the authority constructing and managing them. The cost of building and maintaining the imperial roads is derived from the national funds, the cost of provincial roads from provincial funds; district roads from district funds. The cost of community roads is borne by the several communities interested, aided in certain cases from the district funds. For the imperial and provincial roads the best of engineering skill is employed, while, for immediate repairs, road keepers are at work constantly.

Italian roads are under the supervision of the Minister of Public Works, and are national, provincial, communal, or vicinal, according to the source from which taxes for construction and maintenance are derived.

The more important roads of Denmark are controlled by the county councils, but are subject to the annual inspection of a state engineer; the roads of lesser importance are governed by the parish or township councils.

The main roads of Belgium, those routes running from one part of the kingdom to another, are controlled and managed by the state; another class, provincial roads, are controlled by the province; a third class, communal roads, are controlled by the communal authorities. The construction of these roads is entrusted to a corps of engineers.

In the Netherlands, a network of roads, providing convenient travel from one part of the country to another, is maintained by the general government; other roads are at the expense of the various provinces and communities benefited.

The federal government of Switzerland controls a few of the important roads, but in the main they are built and maintained by the cantonial government through whose territory they pass. The construction and repair of roads of lesser importance pertains to the several townships through which they pass.

Spain, decayed and tottering, the vestiges of an ancient magnificence falling from her, has not joined the good roads movement; nor has Turkey, the home of barbarism. Russia, too, has been exceedingly backward in road building, and as a result her extensive and rich dominions are still practically undeveloped.

These instances of progressive road legislation indicate that the Provinces of Canada are very inadequately equipped with the means of building and maintaining roads in a manner suited to their importance as a public work. Until the relation of the civil engineer to road control is drawn much closer, there can be little

prospect of improvement. This can only be accomplished by inaugurating proper systems of control, in which the importance of roads is more fully recognized by government and citizen. Undoubtedly popular feeling is approaching such an end in many parts of the Dominion. The Premier of Ontario has expressed the willingness of the Government to appropriate $1,000,000, under a system of state aid, but the details of the plan under which this appropriation will be made have not been completed. The general outline is expected to be a central advisory bureau, coupled with the county system, thereby providing for the improvement of main roads under competent engineering supervision. What may be expected from other provinces in the future can only be inferred from the fact that in every one, from Nova Scotia of the East, to British Columbia, of the West, there are prominent and public spirited citizens active in urging measures of road reform, a movement in which the civil engineers of the Dominion should be most capable of taking a part.

## DISCUSSION.

Mr. W. J. Sproule.

Mr. Sproule asked if any attempt had been made to introduce legislation in any of the provinces whereby a re-location of the main concession roads throughout the country could be made?

Mr. W. A. Campbell.

Mr Campbell in reply, said that no attempt had been made to introduce legislation in any of the provinces whereby a re-location of the main roads throughout the country could be made. The importance of this matter was clearly apparent in every community where the roads, as a general thing, had followed the fixed lines of survey, regardless of the topography of the country. Many of those roads had been gravelled or macadamized, and were in good condition, except this one great defect, of steep grades and swampy ground, where a slight deviation would decrease the grade, and lessen the cost of maintenance.

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Thursday, 8th November.

DR. H. T. BOVEY, President, in the Chair.

Arrangements for the Annual Meeting were discussed.

The discussion of Mr. Campbell's paper on "Road Construction and System of Control," occupied the evening.

Thursday, 22nd November.

DR. H. T. BOVEY, President, in the Chair.

Messrs. E. A. Rhys-Roberts, J. G. Kerry and F. H. White, having been appointed scrutineers of the ballot for the election of Members, reported the following elected:—

MEMBER.

R. BICKERDIKE, JR.

ASSOCIATE MEMBERS.

R. J. PARKE, J. K. SCAMMELL,
W. H. SHILLINGLAW, T. H. WIGGINS.

STUDENTS.

W. A. DUFF, H. C. GROUT.

*Paper No. 154.*

## THE MINERS' INCH AND THE DISCHARGE OF WATER THROUGH VARIOUS ORIFICES UNDER LOW HEADS.

BY THOS. DRUMMOND, B.A. SC., A.M. CAN. SOC. C. E.

The water determinations here recorded were made in the Hydraulic Laboratory, McGill University, Montreal.

This laboratory is unusually well equipped. It is fully provided with the modern scientific apparatus necessary for experimental work in all branches of hydraulics.

*General Description.* The laboratory is 39 feet long, 31 feet wide, and contains the following:—An experimental tank and flume; a weir with adjustable widths; a number of large orifices; a battery of five measuring tanks of large capacity; smaller measuring vessels of various capacities of from ten to one hundred gallons; standard gallon, quart and litre measures; a large experimental pump, weighing 55,000 pounds, with interchangeable valves and other special apparatus for experimental work; a 16-inch Pelton wheel, with brake attachments; a turbine tester of special design; an experimental centrifugal pump; an hydraulic ram; apparatus for testing hose up to 800 pounds' pressure per square inch, in lengths of 50

feet; Venturi, Piston and Rotary Meters; hook gauges and chronographs; Darcy's Improved Pitot tubes; metal plane tables, with adjustable feet, for use with the movable measuring vessels; a fixed mercury column, 27 feet high, and several smaller movable columns; a set of pipes for use in determining the resistance to flow due to bends, etc.; a set of carefully made nozzles, with pressure gauge attachments at each end; a set of hydrostatic gauges, with a wide range of pressure; apparatus for measuring the impact of water; apparatus for determining the coefficient of velocity, etc.; jet measuring apparatus; a Rogers' linear comparator; apparatus for studying the inversion of the vein; a large series of standard orifices; special thermometers, scales, verniers and micrometers; glass tanks for illustrating ring motion, critical velocity and stream line phenomena; glass vessels for demonstrating circular and spiral vortex motion, etc., and various other appliances too numerous to mention.

The pumps and shafting in the laboratory are driven by an electric motor. The water is drawn from the city high level reservoir, and is under a pressure of 120 pounds per square inch.

*Detailed Description of the Apparatus used.* The experimental tank is of cast iron, and is 28 feet high and 5 feet square. All flanges are on the outside, and the inside walls are perfectly flush. The water is admitted into a chamber extending right across the bottom of the tank, which contains perforations through which the water flows to the bottom, and is there deflected upward, and passes through two baffle plates with perforated holes, the first being 12 inches and the second 18 inches above the bottom of the tank. To equalize the flow of water as much as possible, the pitch of the holes in the upper plate was determined by the projections on a horizontal plane of equal distances on a sphere of 10 feet diameter, whose centre coincides with the centre of the orifice.

The tank is provided with various inlet and outlet pipes of different sizes, so that the admission and discharge of varying amounts of water are completely governed.

The head of water is shown by a glass gauge 1½ inches diameter, with a graduated brass scale, both of which extend from top to bottom of tank, the zero on the scale being the centre of the orifice A sliding carrier with a horizontal wire moves up and down, and any required head is obtained by bringing the required scale division, the bottom of the meniscus of the water in the tube, the horizontal wire and its reflection in a mirror behind into the same horizontal plane.

A second indicator consists of a float attached to a silk fishing line, passing over a pulley on the top of the tank, and then vertically downward in front. It has a plumbob at the lower end to keep it taut, and an adjustable pointer and stationary graduated

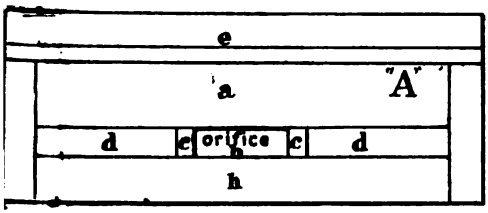
scale. The inflow and outflow are first roughly regulated and the final adjustment is then made by means of small pipes and valves. In this way the operator has the water fully under control and observation, and a constant head can be maintained for any required time when the water has once steadied down to a regular flow.

The orifices, when used in the tank, are centred in a circular ring, which fits into a specially designed circular valve, with gun metal bearing surfaces, forming a water-tight joint, and the centre of the orifice always coincides with the zero of the scale. The valve is provided with a screw adjustment for fixing the verticality of the sides of the orifice, and a handle by which the orifice can be opened or shut. By means of a special artifice the orifice can be changed without lowering the head.

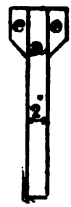
The base of the experimental tank is on a level with the bottom of, and discharges into, an iron flume 35 feet long, 5 feet wide, and 6 feet deep. It has a clear depth below the bottom of the orifice of 3½ feet. It is provided with movable baffle boards to steady the flow of water, and hook gauges, reading to the thousandth of an inch, for measuring the head. At the lower end it discharges over a weir or through an orifice, and either runs to waste or into a battery of five iron measuring tanks, each holding 1,100 gallons, the united measuring capacity being 882.5 cubic feet. These tanks are carefully calibrated, and can be used either together, separately, or in any combination. The flume is provided with a waste gate. When open the water runs to waste, and when closed into the measuring tanks. The gate is electrically connected with a chronograph, which can be read to the 1-100 of a second, so that the beginning and end of an observation is automatically recorded. A good stop watch was used to check the chronograph time.

When orifices are used in the side of the experimental tank the water discharges through a switch, either to waste or into a carefully calibrated 100 gallon cylindrical measuring tank, which rests on a plane table. This switch is also electrically connected with the chronograph, and is used as above.

The wooden orifice used in these experiments was adjustable, and it was arranged as follows:—




Side Elevation




End Elevation

A is a wooden frame 2 inches thick, planed and painted, and made the full width of the flume into which it fits; a is a sliding gate, which slips down between two side pieces, e, the lower surface, being varnished and square edged. This rests on dimension blocks c and c, placed at the proper width to make the required orifice b. It is then clamped into place, the lower surface of the orifice being also square edged and varnished. The voids d and d are then filled in with wooden blocks, and the cracks tallowed and made watertight. The blocks c and c as used at first were made of hardwood, and were set end on, but they were subsequently replaced by brass blocks, as it was found that the wood blocks swelled slightly in the water. The orifice was then carefully measured with Brown & Sharpe's vernier micrometers and also with scales and calipers. The width being taken at the top and bottom, and the height at from 3 to 11 points depending on the width, the means of these gave the height and width of the orifice, which had been carefully levelled. This was done for each set of observations and head, and for every change of orifice.

The word gallon as used in this paper means Imperial gallon.

All heads are measured from centre of the orifice.

The value of g (=32.176) used was determined for Montreal by Commandant Desforges in 1893.

The observations were taken in sets of from 2 to 5, depending upon the agreement and the time available.

The formula $Q = \frac{2}{3} cb\sqrt{2g}\,(H_1^{\frac{3}{2}} - H_2^{\frac{3}{2}})$ was used throughout.

Q=The discharge in cubic feet per second.

C=Coefficient of discharge.

b=Breadth of orifice.

$H_1$=Head from bottom of orifice. } All in feet.

$H_2$=Head from top of orifice.

The discharges here recorded were made under low heads of from 6 to 12 inches, and with two kinds of orifices, viz.:

(1) Standard sharp-edged rectangular orifices in brass from 1 to 4 square inches in area.

(2) Square-edged rectangular orifices in wood, 2 inches thick, 2 to 4 inches in height, and ½ to 24 inches in width.

Condensed tabular statement of the observed discharge Q. The calculated coefficient of discharge C in various orifices with full contraction. Wooden orifices in 2 inch plank planed and painted square edges.

$$Q = \tfrac{2}{3}\, c\, b \sqrt{2g}\ (H_1^{\frac{3}{2}} - H_2^{\frac{3}{2}}),\ g = 32.176.$$

| Average number of cubic feet in run. | Number of observations. | Number. | Temperature Fahrenheit. | Orifice dimensions in inches. Wide. | High. | Actual head in inches above centre of orifice. | Average time Seconds. | Mean observed discharge Q in cubic feet per second. | Coeff. of discharge C Fractional orifice. | Corrected coeff. of discharge C. Even orifice. | Number of Miners ins. of 1.50 cubic feet per minute. Even orifice. | Width side of orifice to side of flume, inches. | Nominal head in ins. above centre of orifice. | Remarks. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 747.85 | 4 | 1 | 45°–50° | 24.00 | 2.0167 | 6.9916 | 573.8 | 1.303169 | .6337 | .6333 | 51.6948 | 18.5 | 7 | Stream slightly irregular |
| 349.09 | 4 | 2 | " | 18.00 | 2.01 | 6.9950 | 361.5 | 0.965495 | .6284 | .6277 | 38.4276 | 21.5 | 7 | " " |
| 164.24 | 5 | 3 | " | 11.992 | 2.01 | 6.9950 | 256.1 | 0.641266 | .6265 | .6258 | 25.5396 | 24.5 | 7 | Steady stream. |
| 150.14 | 5 | 4 | " | 8.96 | 2.01 | 6.9950 | 313.8 | 0.478439 | .6248 | .6249 | 19.1268 | 26.0 | 7 | " " |
| 148.16 | 5 | 5 | " | 6.005 | 2.005 | 6.9975 | 464.8 | 0.318784 | .6212 | .6228 | 12.7092 | 27.5 | 7 | " " |
| 147.80 | 5 | 6 | " | 4.000 | 2.01 | 6.9950 | 701.3 | 0.210758 | .6170 | .6166 | 8.3884 | 28.5 | 7 | " " |
| 147.51 | 2 | 7 | " | 4.000 | 2.01 | 6.9950 | 726.4 | 0.203035 | .6174 | .6165 | 8.0808 | 28.5 | 6.5 | " " |
| 37.94 | 4 | 8 | " | 1 | 2 | 7 | 714.3 | 0.053105 | .... | .6246* | ..... | 30.0 | 7 | Orifice runs full at times |
| 28.73 | 3 | 9 | " | 2 | 1 | 7 | 619.7 | 0.053325 | .... | .6268* | ..... | 29.5 | 7 | " " |
| 20.63 | 2 | 10 | " | 2 | 1 | 7 | 317.6 | 0.064927 | .... | .7632* | ..... | 29.5 | 7 | " full all the time |
| 15.63 | 5 | 11 | " | ½ | 2 | 8 | 436.4 | 0.035782 | .... | .7871* | ..... | 30.25 | 8 | " " |
| 15.66 | 1 | 12 | " | ½ | 2 | 7 | 470.7 | 0.033280 | .... | .7829* | ..... | 30.25 | 7 | " " |
| 804.20 | 2 | 13 | 48 | 24.02 | 3.9924 | 7.0038 | 319.4 | 2.517665 | .6220 | .6191 | 100.8092 | 18.5 | 7 | Stream a little irregular |
| 806.67 | 4 | 14 | 47 | 17.95 | 3.993 | 7.0035 | 428.9 | 1.880923 | .6187 | .6188 | 75.5792 | 21.5 | 7 | " " " |
| 822.19 | 2 | 15 | 47 | 11.98 | 3.9925 | 7.0038 | 658.4 | 1.248712 | .6155 | .6156 | 50.1196 | 24.5 | 7 | Steady stream water backs |
| 821.69 | 2 | 16 | 47 | 11.98 | 3.9925 | 7.0038 | 656.5 | 1.251631 | .6169 | .6171 | 50.2368 | 24.5 | 7 | up a little |
| 291.79 | 5 | 17 | 47 | 6 | 4 | 7 | 469.7 | 0.621116 | .6103 | .6103 | 24.8446 | 27.5 | 7 | " water does |
| 330.47 | 3 | 18 | 45.5 | 4.103 | 3.993 | 7.0035 | 738.5 | 0.412610 | .5964 | .5941 | 16.1184 | 28.5 | 7 | " " [not back up |

The same statement for wooden orifices 2 inches thick at head to give as nearly as possible a discharge of 1½ cubic feet per minute.

| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 663.56 | 4 | 19 | 37.3 | 5.9875 | 2.028 | 6.236 | 539.4 | 0.30321 | .6233 | .6216 | 11.9858 | 27.5 | 6.25 | Steady stream |
| 337.62 | 9 | 20 | 37.0 | 12.000 | 2.026 | 6.237 | 385.6 | 0.614095 | .6298 | .6282 | 24.2485 | 24.5 | 6.25 | " " |
| 419.73 | 3 | 21 | 37.5 | 17.995 | 2.03 | 6.235 | 345.7 | 0.923017 | .6313 | .6289 | 36.3851 | 21.5 | 6.25 | Stream steady water pul- |
| 387.75 | 4 | 22 | 36.0 | 23.98 | 2.025 | 6.2375 | 397.3 | 1.23135 | .6320 | .6312 | 48.6865 | 18.5 | 6.25 | " " " " [sates |
| 819.82 | 5 | 23 | 36.0 | 3.996 | 4.015 | 6.2425 | 814.4 | 0.39259 | .6132 | .6128 | 15.6998 | 28.5 | 6.25 | " " " " |
| 424.50 | 4 | 24 | 36.0 | 5.995 | 4.015 | 6.2425 | 549.6 | 0.590386 | .6136 | .6130 | 23.5560 | 27.5 | 6.25 | Steady stream |
| 383.59 | 4 | 25 | 35.5 | 11.985 | 4.023 | 6.2385 | 407.3 | 1.187446 | .6154 | .6151 | 47.2853 | 24.5 | 6.25 | " " |
| 150.56 | 3 | 26 | 35.0 | 17.990 | 4.034 | 6.233 | 357.7 | 1.304959 | .6222 | .6213 | 71.6296 | 21.5 | 6.25 | " " water pulsates |
| 213.84 | 3 | 27 | 36.0 | 25.462 | 4.044 | 6.228 | 317.5 | 2.56289 | .6230 | .6218 | 101.5495 | 17.75 | 6.25 | " " " " |

The orifice ½ x 2 was tested in a horizontal and vertical position with heads of from 6–12 inches and runs full throughout.

* See pages 140 and 141 for notes on these orifices.

Condensed tabular statement of the observed discharge Q. The calculated coefficient of discharge C in various orifices with full contraction. Standard brass orifices with sharp edges.

$$Q = \tfrac{2}{3}\, c\, b\, \sqrt{2g}\, \left(H_1^{\frac{3}{2}} - H_2^{\frac{3}{2}}\right),\ g = 32.176.$$

| Average number of gallons in the run. | Number of observations. | Number. | Temperature Fahrenheit. | Orifice dimensions in inches. Wide. | High. | Head in inches above centre of orifice. | Average time. Seconds. | Mean observed discharge Q in cubic feet per second. | Coeff. of discharge C. | REMARKS. |
|---|---|---|---|---|---|---|---|---|---|---|
| 98.739 | 5 | 28 | 37.7 | 2 | 2 | 12 | 116.5 | 0.135985 | .6104 | Not so regular, but fairly. |
| 98.710 | 3 | 29 | 37.1 | 2 | 2 | 10 | 127.5 | 0.124265 | .6113 | " " good |
| 98.260 | 4 | 30 | 41.0 | 2 | 2 | 8 | 141.5 | 0.111451 | .6130 | " " |
| 98.318 | 4 | 31 | 38.2 | 2 | 2 | 7 | 151.1 | 0.104419 | .6142 | " " |
| 98.161 | 3 | 32 | 38.7 | 2 | 2 | 6 | 162.9 | 0.096743 | .6147 | " " |
| 97.336 | 3 | 33 | 40.3 | 2 | ½ | 6 | 625.8 | 0.024983 | .6339 | Steady stream. |
| 97.418 | 3 | 34 | 39.9 | 2 | ½ | 7 | 579.6 | 0.028971 | .6338 | " " |
| 97.616 | 3 | 35 | 39.7 | 2 | ½ | 8 | 544.6 | 0.028761 | .6323 | " " |
| 97.099 | 3 | 36 | 46.2 | 2 | ½ | 10 | 487.7 | 0.031944 | .6282 | " " |
| 97.382 | 3 | 37 | 44.1 | 2 | ½ | 12 | 467.1 | 0.034949 | .6273 | " " |
| 97.574 | 4 | 38 | 41.7 | ½ | 2 | 12 | 448.1 | 0.034887 | .6265 | Steady stream. |
| 97.434 | 3 | 39 | 45.3 | ½ | 2 | 10 | 489.6 | 0.031916 | .6279 | " " |
| 97.767 | 3 | 40 | 41.2 | ½ | 2 | 8 | 546.6 | 0.028734 | .6321 | " " |
| 97.571 | 3 | 41 | 41.1 | ½ | 2 | 7 | 581.4 | 0.026931 | .6335 | " " |
| 97.307 | 3 | 42 | 41.2 | ½ | 2 | 6 | 625.0 | 0.024981 | .6354 | " " |
| 97.615 | 3 | 43 | 39.5 | 2 | 1 | 12 | 228.4 | 0.068577 | .6153 | Steady stream. |
| 97.733 | 3 | 44 | 46.5 | 2 | 1 | 10 | 249.6 | 0.062708 | .6166 | " " |
| 97.602 | 3 | 45 | 39.2 | 2 | 1 | 8 | 278.8 | 0.056171 | .6176 | " " |
| 97.639 | 4 | 46 | 39.2 | 2 | 1 | 7 | 297.3 | 0.052699 | .6194 | " " |
| 97.377 | 5 | 47 | 41.6 | 2 | 1 | 6 | 319.4 | 0.048925 | .6212 | " " |
| 97.362 | 5 | 48 | 38.6 | 1 | 2 | 6 | 320.1 | 0.048812 | .6203 | Steady stream. |
| 97.581 | 3 | 49 | 42.9 | 1 | 2 | 7 | 297.6 | 0.052609 | .6188 | " " |
| 97.860 | 3 | 50 | 39.5 | 1 | 2 | 8 | 278.8 | 0.056325 | .6196 | " " |
| 97.321 | 3 | 51 | 44.1 | 1 | 2 | 10 | 249.0 | 0.062827 | .6180 | " " |
| 97.593 | 3 | 52 | 38.8 | 1 | 2 | 12 | 228.2 | 0.068751 | .6172 | " " |
| 97.573 | 4 | 53 | 41.4 | 4 | ½ | 6 | 312.1 | 0.050157 | .6372 | Slightly irregular. |
| 97.928 | 3 | 54 | 41.2 | 4 | ½ | 7 | 289.8 | 0.054223 | .6372 | " " |
| 97.648 | 2 | 55 | 43.7 | 4 | ½ | 8 | 271.4 | 0.057724 | .6344 | " " |
| 97.617 | 4 | 56 | 38.5 | 4 | ½ | 10 | 242.8 | 0.064502 | .6342 | " " |
| 97.625 | 4 | 57 | 41.3 | 4 | ½ | 12 | 222.9 | 0.070309 | .6310 | " " |
| 97.477 | 5 | 58 | 43.5 | ½ | 4 | 12 | 221.9 | 0.070518 | .6336 | Slightly irregular. |
| 97.692 | 4 | 59 | 40.6 | ½ | 4 | 10 | 242.9 | 0.064546 | .6357 | " " |
| 97.672 | 6 | 60 | 45.0 | ½ | 4 | 8 | 271.4 | 0.057740 | .6364 | " " |
| 97.851 | 3 | 61 | 42.9 | ½ | 4 | 7 | 290.2 | 0.054098 | .6380 | " " |
| 97.520 | 4 | 62 | 43.4 | ½ | 4 | 6 | 311.4 | 0.050248 | .6408 | " " |
| 97.192 | 8 | 63 | 43.5 | 1 | 1 | 12 | 454.9 | 0.034287 | .6152 | Steady stream. |
| 97.119 | 3 | 64 | 43.4 | 1 | 1 | 10 | 496.7 | 0.031379 | .6171 | " " |
| 97.358 | 3 | 65 | 46.3 | 1 | 1 | 8 | 561.1 | 0.028005 | .6177 | " " |
| 97.422 | 3 | 66 | 43.3 | 1 | 1 | 7 | 664.4 | 0.025447 | .6193 | " " |
| [illegible] | 2 | 67 | 43.3 | 1 | 1 | 6 | 687.3 | 0.024447 | .6208 | " " |

The brass orifices were carefully measured at a temperature of 69° F.

1 × 1 = 0.9962 × 0.9967 = 0.99302 square inches
½ × 2 = 0.5008 × 1.9967 = 0.99947 " "
1 × 2 = 1.9983 × 1.000 = 1.9983 " "
2 × 2 = 1.99925 × 1.99924 = 3.99849 " "
½ × 4 = 0.5005 × 3.9988 = 2.00149 " "

Coefficients of discharge for various standard brass orifices of various sizes with various heads.

| Orifice dimensions inches. Wide. | High. | Head inches | Coefficient of discharge C | Orifice dimensions inches Wide. | High. | Head inches | Coefficient of discharge C | Orifice dimensions inches. Wide. | High. | Head inches | Coefficient of discharge C |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1 | 6 | .6208 | 2 | ½ | 8½ | .6308 | 2 | 1 | 11 | .6156 |
| 1 | 1 | 6½ | 201 | 2 | ½ | 9 | 6302 | 2 | 1 | 11½ | .6154 |
| 1 | 1 | 7 | 194 | 2 | ½ | 9½ | .6297 | 2 | 1 | 12 | .6153 |
| 1 | 1 | 7½ | 188 | 2 | ½ | 10 | .6292 | 4 | ½ | 6 | .6382 |
| 1 | 1 | 8 | .183 | 2 | ½ | 10½ | .6287 | 4 | ½ | 6½ | .6371 |
| 1 | 1 | 8½ | .6179 | 2 | ½ | 11 | 6283 | 4 | ½ | 7 | .6361 |
| 1 | 1 | 9 | .1 4 | 2 | ½ | 11½ | .6278 | 4 | ½ | 7½ | .6352 |
| 1 | 1 | 9½ | .6171 | 2 | ½ | 12 | .6274 | 4 | ½ | 8 | .6344 |
| 1 | 1 | 10 | .6166 | ½ | 2 | 6 | .6359 | 4 | ½ | 8½ | .6338 |
| 1 | 1 | 10½ | .6163 | ½ | 2 | 6½ | .6347 | 4 | ½ | 9 | .6332 |
| 1 | 1 | 11 | .6159 | ½ | 2 | 7 | .6336 | 4 | ½ | 9½ | .6327 |
| 1 | 1 | 11½ | .6156 | ½ | 2 | 7½ | .6325 | 4 | ½ | 10 | .6323 |
| 1 | 1 | 12 | .6152 | ½ | 2 | 8 | .6315 | 4 | ½ | 10½ | .6319 |
| 2 | 2 | 6 | .6151 | ½ | 2 | 8½ | .6306 | 4 | ½ | 11 | .6316 |
| 2 | 2 | 6½ | .6145 | ½ | 2 | 9 | .6298 | 4 | ½ | 11½ | .6313 |
| 2 | 2 | 7 | .6139 | ½ | 2 | 9½ | .6290 | 4 | ½ | 12 | .6310 |
| 2 | 2 | 7½ | 6134 | ½ | 2 | 10 | .6284 | ½ | 4 | 6 | .6408 |
| 2 | 2 | 8 | 6130 | ½ | 2 | 10½ | 6278 | ½ | 4 | 6½ | .6392 |
| 2 | 2 | 8½ | 6126 | ½ | 2 | 11 | .6274 | ½ | 4 | 7 | .6380 |
| 2 | 2 | 9 | .6122 | ½ | 2 | 11½ | .6269 | ½ | 4 | 7½ | .6370 |
| 2 | 2 | 9½ | 6118 | ½ | 2 | 12 | .6265 | ½ | 4 | 8 | .6364 |
| 2 | 2 | 10 | 6116 | 2 | 1 | 6 | .6212 | ½ | 4 | 8½ | .6359 |
| 2 | 2 | 10½ | .6112 | 2 | 1 | 6½ | .6202 | ½ | 4 | 9 | .6354 |
| 2 | 2 | 11 | .6110 | 2 | 1 | 7 | .6193 | ½ | 4 | 9½ | .6350 |
| 2 | 2 | 11½ | .6107 | 2 | 1 | 7½ | .6185 | ½ | 4 | 10 | .6346 |
| 2 | 2 | 12 | .6104 | 2 | 1 | 8 | .6178 | ½ | 4 | 10½ | .6343 |
| 2 | ½ | 6 | .6352 | 2 | 1 | 8½ | .6172 | ½ | 4 | 11 | .6341 |
| 2 | ½ | 6½ | 6341 | 2 | 1 | 9 | .6169 | ½ | 4 | 11½ | .6338 |
| 2 | ½ | 7 | .6330 | 2 | 1 | 9½ | .6164 | ½ | 4 | 12 | .6336 |
| 2 | ½ | 7½ | .6322 | 2 | 1 | 10 | .6161 | | | | |
| 2 | ½ | 8 | .6314 | 2 | 1 | 10½ | .6158 | | | | |

The brass standard orifices were placed in the valve in the experimental tank, which has adready been described, and in all the experiments the water ran through the switch into the 100 gallon tank, fitted with a glass gauge tube and a scale graduated to gallons.

The tubes are of large size; they were always kept wet, and the scales were graduated by reading the bottom of the meniscus under these conditions of calibration. Capillary action is thus eliminated.

The observed discharges from the brass orifices rarely differed by

more than 2-1000 of a cubic foot per minute, and they often agreed almost exactly.

The correct size of the brass orifices, as measured at a temperature of 65 degrees F., is given elsewhere in the paper.

The discharges from the wooden orifices were observed at the lower end of the flume, and measured in the large tank. The agreement was not so close as was anticipated with the larger quantity of water.

With the 12, 18 and 24 inch orifices, vertices were formed, and there was an appreciable velocity of approach. No corrections were applied, as their effect would be very slight, and practically nil.

With these same orifices there was a slight breaking up of the water, and the clearance was not as good as it might have been. From this cause the discharge for these orifices is probably slightly less than it should be. This was tested in the case of the 12 x 4 inch orifice, and it gave a slightly increased discharge, amounting to about 3-1000 of a cubic foot per second.

With two of the largest orifices there was a pulsation in the water affecting the head. In such cases the mean head from a number of readings was adopted.

Regarding the wooden orifices, it can be said that wood is not an absolutely stable material. The wood, however, was made practically waterproof, and any change that may have occurred from swelling was inappreciable in the interval of time necessary for one set of observations.

No attempt was made to get exact inch dimensions for the orifice; it was levelled, set, and measured as described above.

Full contraction of the vein was maintained throughout, except in particular cases referred to later on.

Great care was taken to make and keep the edges square and sharp, and, this being done, the thickness of the orifice within certain limits makes little or no difference.

The large jets show more irregularity and disturbance in the stream lines. Corrugations appear on the surface, and they undulate and move slightly.

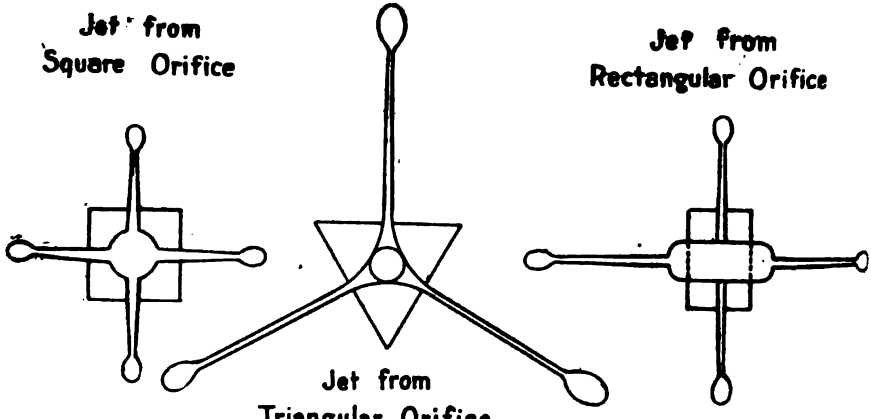
The shape of the orifice has a perceptible effect upon the discharge. Circular orifices give the least discharge, rectangular orifices the greatest, and square orifices are intermediate. As the rectangular orifice becomes wider, the depth being the same, it will discharge proportionately more water, and the coefficient of discharge $C$ becomes greater. $C$ increases as the depth decreases, the width remaining constant, and it decreases as the size of the orifice increases, the same shape being maintained. $C$ has a smaller value for large than for small orifices, and for the same orifice it decreases as the head increases.

The curve of coefficients for a particular orifice is a curve resembling the cubical parabola. With great heads it tends to become assymtotic, and the coefficient $C$ probably eventually becomes a constant. Several of these curves are drawn to scale, with the head in inches, and the calculated coefficient as co-ordinates. A curve is then drawn in to accord with these points as nearly as possible, so that the coefficient for any point within its limits can be read off directly.

There are slight discordances, as may be seen, which are due to several causes. First, there is the exaggerated scale, making the difference appear much greater than it really is. Again, the observations were started with heads of 6, 7, 8 and 12 inches, and some of the observations under the 10-inch head were taken with a separate setting of the orifice, but with the same agreement, and the same care as before. A few of these observations for some reason do not agree well, and several have been rejected. The errors of observation are only slight, and would not account for them. They are caused (1) by temperature effects; (2) by certain inherent irregularities in the flow of water under low heads. Temperatures were taken, and an attempt was made to apply corrections by using the recognized coefficients of expansion for iron, brass, copper and water. Theoretically, they can be reduced to a common temperature; practically, however, it is uncertain. In the case of the orifices and tanks, one side of the metal only is in the water, and the air has access to the other. We can only assume that the contained water is of the same temperature as the metal, and this does not appear to be the case. Till more is known, these corrections cannot be applied with certainty.

With the head and observed discharge as co-ordinates, the curve of discharge is practically a parabola, but not exactly so. If it were, the whole theory of hydraulics would be much simplified.

A curious and beautiful phenomenon is the *inversion of the vein*, and as this term has been used, and is not generally described in text-books, a short description may be given. With a carefully made standard circular orifice the jet issues like a clear crystal bar of glass, and seems to be perfectly rigid. With other orifices of three or more sides, there is a remarkable change. The jet develops wings or rays in certain wave lengths. The first are at right angles to the edges of the orifice, and the number is the same as the sides. This is because the issuing water from the corners of the orifice has a greater convergent tendency than elsewhere in the jet, and it presses the water out into these rays. All the brass orifices show this phenomenon well, and the wooden ones show it to a lesser extent, depending upon the size. Several examples are given.




The observations here recorded should be especially useful in British Columbia as furnishing data for the delivery of water at mines, and this leads to the question of the miner's inch of water.

*The Miner's Inch of Water*, it may be explained, is an arbitrary module adopted in mining districts for selling water. It is variously defined as being the amount of water discharged by an orifice one inch square, or an equivalent fraction of a larger orifice with a head of from 6 to 9 inches. The thickness of the orifice is usually 2 inches.

One great difficulty is that it is a variable quantity depending upon the specified head, and therefore all such modules should also define the flow in cubic feet per minute.

In British Columbia it is defined as being 1.68 cubic feet of water per minute, or that quantity of water which will pass through an orifice ½ an inch wide, 2 inches high and 2 inches thick, with a constant head of 7 inches above the top of the orifice, and every additional inch shall mean so much as will pass through the said orifice extended horizontally ½ an inch. As a definition, unfortunately, this is wrong. In the first place, widening the orifice as above changes the coefficient of discharge, and therefore the discharge itself. In the second place, this orifice actually discharges 2.147 cubic feet of water per minute, instead of 1.68 cubic feet, and this brings out a curious point referred to above, that certain small orifices, with a thickness of 2 inches run full like a short tube, the vein is not contracted, and they actually give a greater discharge than they should give. The 1 x 2, ½ x 2 and 2 x 2 inch orifices run under these conditions, and the 1 x 1 inch orifice is also the same, though it was not actually tried. The ½ x 2 inch orifice runs full.

The 1 x 2 inch orifice 2 inches thick is just on the margin between

flow with contraction and full bore. If fixed in the vertical position, with longest diameter vertical, the vein contracts. If fixed in the horizontal position, with the longest diameter horizontal, it will also contract, but if rubbed with the fingers on the edge it will run full for a time and then contract again. If kept running full in this way it will discharge about 1 cubic foot of water per minute more than when full contraction takes place.

The 2 x 2 inch orifice runs partly full, that is to say, the lowest half of the orifice where the issuing vein curves down, runs full, while the upper half contracts. This largely increases both the discharge and the coefficient, the flow becomes irregular, and it makes it practically impossible to determine the value of a single Miner's Inch when the orifice is 2 inches think. For this reason 1-12th of the flow from the 6 x 2 inch orifice was chosen as the standard for the unit Miner's inch.

There are practical difficulties in the way of delivering absolutely exact quantities of water, and they cannot be measured out as a pound of tea is weighed over the counter. The definition of the module or unit, however, should be correct within a reasonable limit of error. If it is a definition of a single miner's inch from an orifice of one square inch, it should go no farther. If the inch is defined as being some fractional part of the discharge from a larger orifice, it should go no farther than the capacity of that orifice, and as it is an unknown quantity to the outside world the discharge should be given in cubic feet per minute. Convenient discharges are 1½ and 2 cubic feet. The flow under low heads is irregular. Heads of 1 ft. or more are not convenient because the water is delivered from ditches or flumes where the depth of water is never great. The question thus resolves itself into a choice of a standard module or unit from a flow under two conditions.

(1) With a low head of 6¼ inches above the centre of the orifice, giving a discharge of 1½ cubic feet per minute, with the advantage that it is already practically recognized as the miner's inch, and with the disadvantage that the flow is irregular.

(2) With a head of 11½ inches above the centre of the orifice, and a discharge of 2 cubic feet per minute, the flow being much more regular, but the quantity discharged new to the people.

The author personally favors the first and a definition of the inch is given, including discharges of from one to one hundred Miner's Inches.

Definition of the Miner's Inch:—

The water taken into a ditch or sluice shall be measured at the ditch or sluice head. It shall be taken from the main ditch, flume or canal through a box or reservoir arranged at the side, and the water shall have no appreciable velocity of approach. The

orifice shall be fixed vertically at right angles to the delivering waterway, and the edges and corners shall be square and sharp, and the top, bottom, and sides of the orifice at right angles to the pressure board. The issuing vein shall be fully contracted, and the discharge shall be freely into air. The distance between the sides and bottom of the waterway, and the sides and bottom of the orifice shall be at least 3 times the deast dimension of the orifice.

One Miner's Inch of water shall mean 1-12th of the quantity which will discharge through an orifice six (6) inches wide and two (2) inches high, made of two inch planks, planed and made smooth. The water shall have a constant head of 6¼ ins. above the centre of the orifice. It shall mean a discharge of 1½ cubic feet per minute.

Discharges up to and including 101.5495 Miner's Inches of 1.50 cubic feet of water per minute shall be as in the following table:

| Dimensions of orifice inches. | | Head in inches over centre of orifice. | Number of miners' inches of 1½ cubic ft. per minute. |
|---|---|---|---|
| Width | Depth | | |
| 6 | 2 | 6.25 | 11.9858 |
| 12 | 2 | 6.25 | 24.2485 |
| 18 | 2 | 6.25 | 36.3851 |
| 24 | 2 | 6.25 | 48.6865 |
| 4 | 4 | 6.25 | 15.6998 |
| 6 | 4 | 6.25 | 23.5560 |
| 12 | 4 | 6.25 | 47.2853 |
| 18 | 4 | 6.25 | 71,6296 |
| 25½ | 4 | 6.25 | 101.5495 |

The actual value of the miner's inch, as taken from the standard adopted is 1.4982 cubic feet of water per minute, which value increases as the orifice widens; 1.500 cubic feet per minute, therefore, is an average differing by very little from the actual average obtained.

As stated above, no attempt was made to reduce the observations to a common temperature. The temperatures for the brass orifice varied between 37.0 degrees and 46.5 degrees F., a range of 9.5 degrees F., with an average temperature of 42 degrees F.

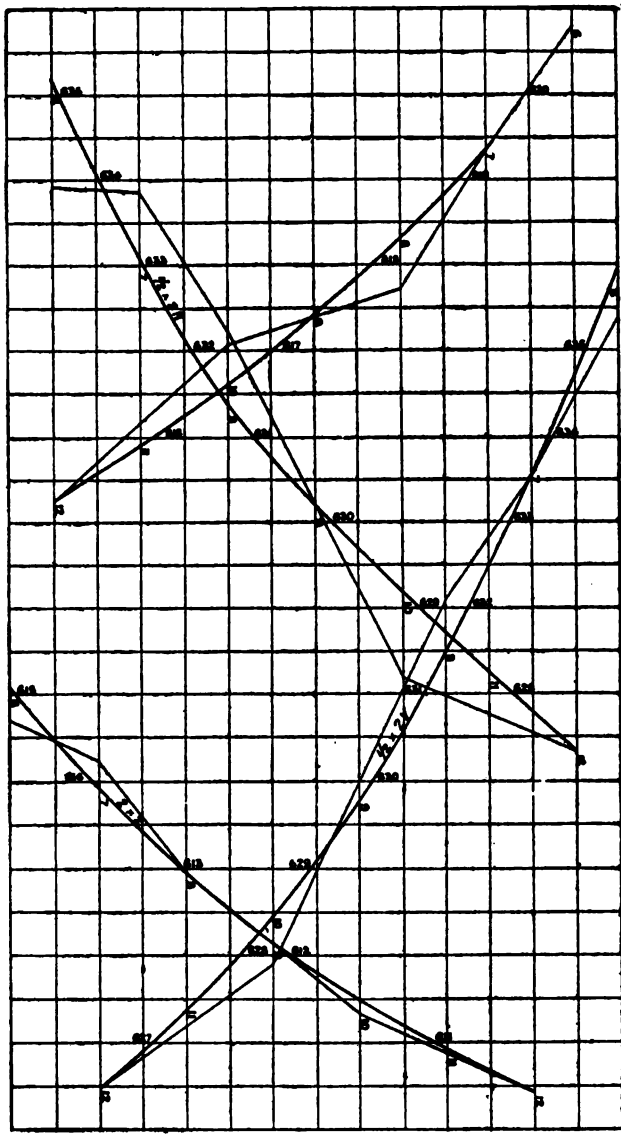
The temperature for the wooden orifices varied between 35 degrees and 50 degrees F., with an average of 45 degrees F.

The mean temperature for the whole was 45 degrees F.

Every precaution was taken to remove disturbing effects, and to make the work as accurate as possible.

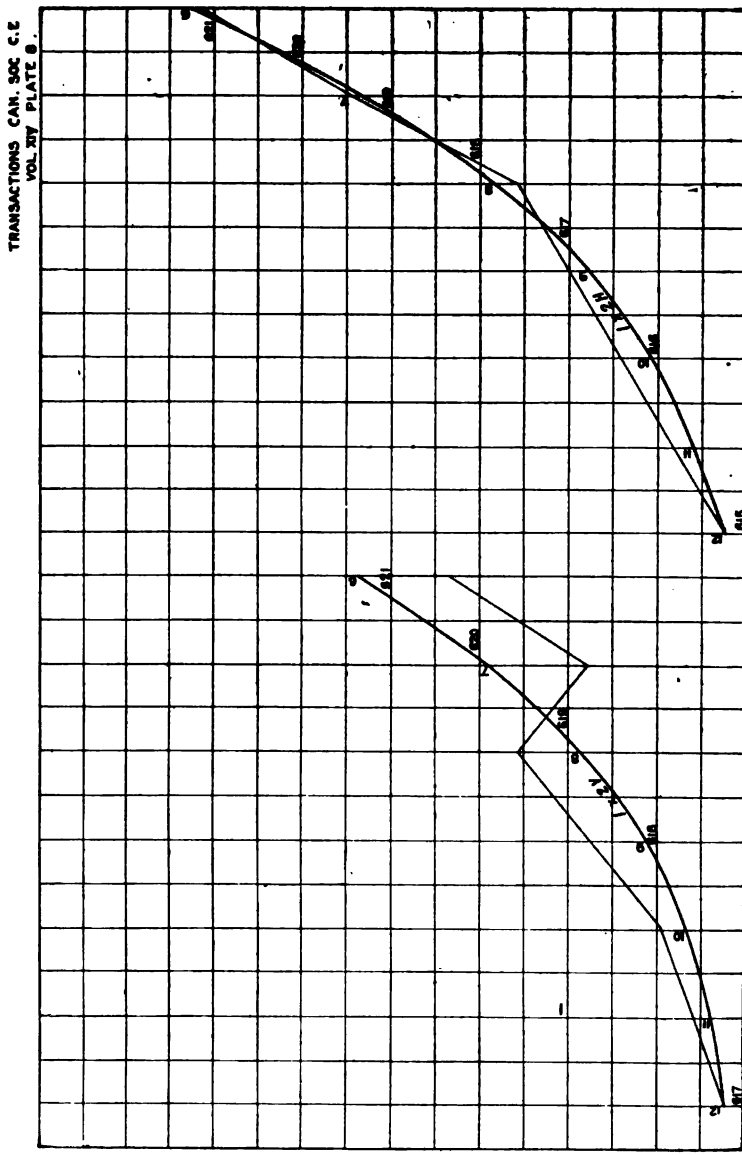
Altogether some 350 observations were made in sets of from 2 to 5, involving a considerable amount of labour, both in the work and calculations.

With a more careful examination of the results it may be possible to give a useful discussion upon the paper.

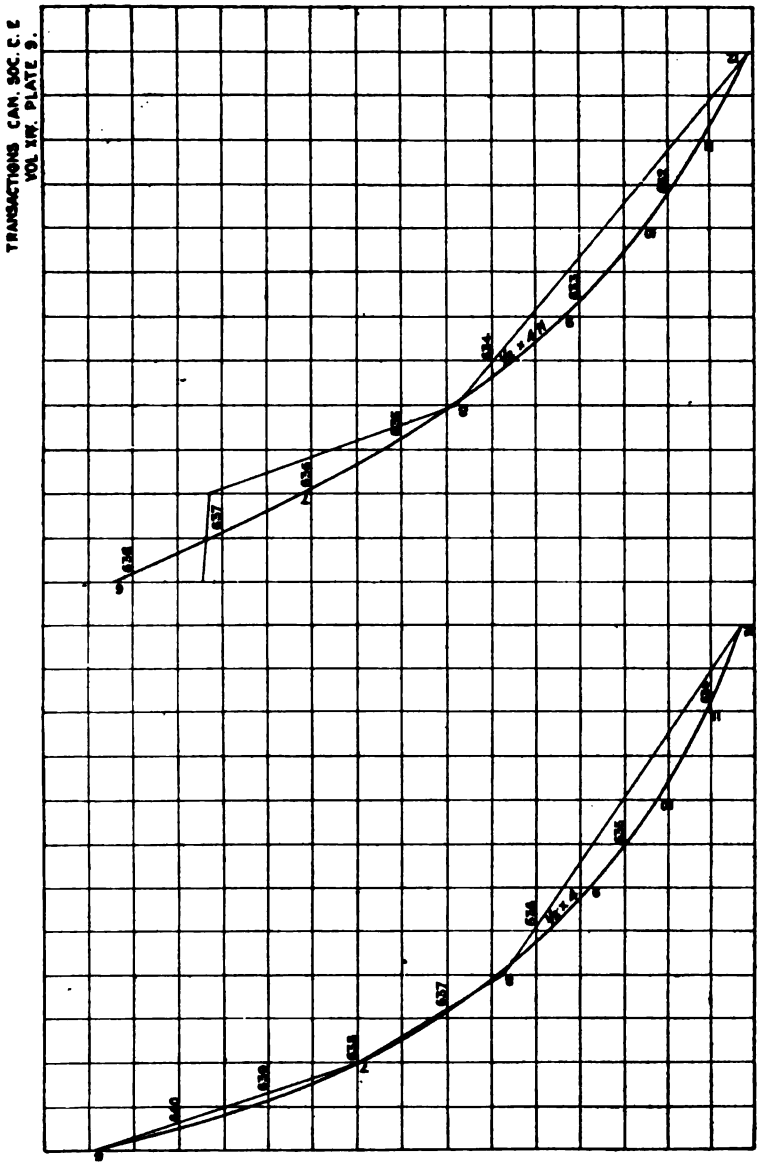


CO-EFFICIENT CURVES

TRANSACTIONS CAN SOC. C.E.
VOL. XII. PLATE 7




CO-EFFICIENT CURVES.


TRANSACTIONS CAN. SOC. C. E
VOL XIV. PLATE 9.
CO-EFFICIENT CURVES.

## DISCUSSION.

Mr. E. Mohun. The profession generally, and the Can. Soc. C. E. in particular, is deeply indebted to the authorities of McGill, and the engineers who devote so much time and trouble to make experiments in the interests of the profession, which are entirely beyond the reach of the ordinary practitioner; and in the paper now before us is given the value of C for the orifice, prescribed by the British Columbia Legislature, a value which the writer has, for some time, been desirous of obtaining.

In 1890 the writer had the privilege of submitting to the Society a paper (Vol. IV., Part 1, Trans.), in which that very eccentric measure, the "Miner's Inch," as used in different localities on the Pacific Slope, was described.

The writer's attempt to induce the Government of British Columbia, some twenty years ago, to make the inch a fixed quantity of water, irrespective of the mode of measurement, failed, but one point was gained, the head and height of the orifice were fixed, instead of being variable quantities, though the definition was still deficient, as was well pointed out by Mr. Butler (Page 165, Vol. IV., Trans.), and by Mr. Drummond in the present paper.

In 1897 the Legislature of British Columbia enacted as follows, (Chap. 190, 1897, Sec. 143):—

"143. In measuring water in any ditch or sluice in the absence of any rules in that behalf established, by Order or Orders in Council, and applied to any record or any class of records, the following rules shall be observed:—The water taken into a ditch or sluice shall be measured at the ditch or sluice head. No water shall be taken into a ditch or sluice except in a trough placed horizontally at the place at which the water enters it, and which trough shall be extended two feet beyond the orifice for the discharge of the water. One inch of water shall mean the quantity that will pass through an orifice two inches high by half an inch wide, made in a two-inch plank, the water to have a constant head of seven inches above the upper side of the orifice, and every additional inch of water shall mean so much as will pass through the said orifice extended horizontally half an inch. In cubic measurement, one inch of water shall mean a flow of water equal to 1.68 cubic feet per minute."

Immediately after the passage of the foregoing, the writer pointed out to the Provincial Secretary that with the orifice described an inch would mean about 25 per cent. more than the quantity, 1.68

cubic feet a minute, mentioned at the end of the section, and suggested that the whole of the clause which refers to the mode of measurement should be repealed, but without effect.

The writer then communicated with President Bovey, who confirmed his view that the orifice would run full bore, and gave, as an expression of opinion in the absence of experiments, that the value of C would approximate 0.8.

It would seem that the only legal method of measurement, except by special Orders in Council, is by the orifice, and the discharge will vary with its width, making possibly differences of 20 or 25 per cent. between narrow and wide orifices, and here the writer feels somewhat at a loss since no values of C are given in the table for wide orifices with 8 in. head.

Returning to the definition, 1.68 cubic ft. a minute is equivalent to 2,419 cubic ft. a day; but with the orifice, the discharge in cubic ft. a day $= c \sqrt{2g}$. h x area x 86,400, which, taking c = O. 7,871 and g = 32,176, is equivalent to 3,093, or over 25 per cent. more than by the second definition.

There are several objections to this mode of measurement, those who have made experiments fully understand the difficulties in maintaining constant heads, and ensuring the true heights and widths of orifices.

Again, in dealing with other than a small quantity of water, such mode of measurement is utterly impracticable. In British Columbia we have as high as 100,000 inches recorded. It would be interesting to watch preparations for measuring this quantity by the prescribed rule; the mind fails to grasp the idea of a level flume over 4,000 feet wide running two inches of water; and imagine the feelings of the shareholders of a company, called upon to grade a mountain gorge to receive such a flume. Joking apart, however, it is presumed that in such a case a special Order in Council would be issued, but as all engineers know, it is by no means an easy matter to decide upon the most accurate and reasonably economical method of guaging considerable volumes of flowing water.

The writer's conclusions are:

1. That the "Miner's Inch" should represent a definite quantity of water, totally independent of any one particular method of measurement, and he does not favour that by orifice and head, which is better adapted to the laboratory than to the field.

2. That the Inch should mean 1.68 cubic feet a minute, which is a very close approximation to his ideal of 100 cubic feet an hour, and would not conflict with vested interests in British Columbia, the only Province of which he can speak with knowledge.

3. That the quantities should be determined by any of the means which have obtained the approval of eminent hydraulicians, whether

by weirs, venturi, or current meters, or Kutter's formula; and though the writer is unable at the present moment to devote the time necessary for investigation he is strongly of opinion that the last method would be eminently adapted for use in open flumes; a portion of which might be, for purposes of measurement, truly guaged and laid. In pipes probably the Venturi Meter would give most satisfactory results.

4. It would appear desirable that the "Miner's Inch" should be uniform throughout the Dominion, and this result can only be obtained by enlisting the sympathies of both the Dominion and Provincial Governments in the matter, which would all have to agree and legislate on the subject.

## IMPERFECT CONTRACTION.

A very general measure in British Columbia is 100 inches, under the existing law, a flume 50 inches wide, with an orifice 2 inches high, with an 8 inch head, would discharge 100 inches, but with that width, the sides of the orifice being in the same plane with the sides of the flume, what would be the value of C? The writer has been unable to deduce a coefficient, and would be grateful if some better authority than himself would furnish one.

The rule appears to be that if the sides of a vessel approach near to the edges of the orifice, they interfere with the convergence of the streams to which the contraction is due, and the contraction is then modified. The writer thinks that it is Greenhill and Unwin who state, that the influence of the sides of the flume begin to be felt if their distance from the edges of the orifice is less than 2.7 times the width of the orifice. It appears to him that if the width of the flume is that of the orifice, there will only be contraction on the upper side; and that in such a case the value of C would be high.

The writer is well aware that the expression "seven inches above the top of the orifice" has been cavilled at, and that "eight inches above the centre of gravity of the orifice" has been suggested as the proper definition; as, however, the two expressions mean exactly the same thing, and the former is more easily understood by the majority of those interested, the writer holds it the better phrase, being one of those who desire to render matters intelligible to such as are not conversant with scientific terms.

No doubt many members of the Society have, like the writer, spent months in the mountains, and to such he would appeal, and without hesitation declare that in the first place, probably far removed from civilization, with the ordinary miner's outfit, it would be practically impossible to construct a flume and orifice which would be accurate in all respects, and in the second place to secure a uniform head of eight inches.

Mr. Chas. Baillairge.

The writer cannot conclude without expressing his deep sense of obligation to the authorities of McGill and the engineers who, like Mr. Drummond, have taken so much trouble to give reliable information upon the subject of discharge through orifices.

Mr. Baillairge had much pleasure and satisfaction in hailing the issue of this paper from such an institution as McGill, so well provided as he knows it to be with all the most modern apparatus applicable to the results arrived at.

As he cannot attend the meeting at which the paper will be discussed, and as the rules of the Society allow of this being done by correspondence, may he request that the question as to how there is such a discrepancy in "co-efficients," say between No. 11 et al., and Nos. 12, 13, 14; for while coefficient to No. 11 is .6168 under an orifice of 2 inches wide by 1 inch high, and a head of 7 inches, No. 12, under same size orifice and same head, gives a coefficient of .7332, differing from that of No. 11 by .1064, while the whole series of other coefficients with the exception of Nos. 13 and 14, range only between .6104 and .6408, or a difference of only .0304, or less by more than three times, than the difference between Nos. 11 and 12.

Will the author please explain the difference between the headings *"steady stream,"* or *"vein runs full."* The writer would now recommend that McGill, through Mr. Drummond, or Mr. Bovey himself, do institute a series of experiments on the loss of *hydraulic* effect on open air jet as due to short and long stretches of pipe of the same small diameter.

The necessity for this is made evident by the results arrived at by the writer in 1885, or after the putting in of the second or new main of 30 inches diameter (the first or 18 inch main laid under G. Baldwin of Boston in 1851). These results are tabulated in the accompanying paper or report, where the effect is seen of the differences of delivery from pipes of the same and of varied diameters and of different lengths.

For instance at No. 30, it is seen that under a static head of 100 lbs., corresponding to a static height of 233 feet, or the height to which water under that head or pressure would rise if enclosed in a pipe; it did not reach one-tenth as high when flowing through 50 feet of 3 inch fire hose and from a 1⅛ inch nozzle, but only to 20 feet, or up to second floor of the woolen factory at the corner of Jacques Cartier and Dorchester streets.

No. 8 of the table shows a greater height of fire or open jet, say 70 feet, or just over apex of roof of St. Bridget's Asylum at corner of Grande Allée and De Salaberry street; thus falling short by 68 feet or 50 per cent of height due to static pressure at that point. But though the length of 4 inch pipe in this case is 1,770 feet, or

greater than that in the former case, it will be noticed that in the latter case the total length of pipe run through from the point of delivery at Mount Pleasant was but 1,770 feet, while, in the former case (No. 30 of tabular statement), the run was not only through the 1,315 feet of 4 inch pipe, but that before reaching this, the water had to run through 3,440 feet of 14 inch pipe, 1,575 feet of 12 inch, 700 feet of 8 inch, and 575 feet of 6 inch pipe, a total distance of 7,605 feet.

The writer believes this tabular statement to be the only one of the kind ever prepared or published for any city that he is acquainted with, and that it contains the most valuable, not theoretical, but true, practical or actual data, whence to predicate the flow of water through varying lengths of pipe of varying diameters, and that as such it should now be reproduced for the information of the profession at large in the forthcoming volume of the Transactions of the Society.

The writer, in expressing a desire that experiments be made at McGill on the comparative flow (as modified by friction) of water through varying lengths of pipe of the same and of small diameter, would suggest, to render the thing possible, on the premises of the College, or without going out of the precincts of the College grounds, that these trials be instituted on pipes of 6 inches diameter and down from 5 inches to 4 inches, 3 inches, 2 inches and 1 inch, in lengths of 100 or 200 to say 2,000 or 3,000 feet, for the 6 inches, 5 inches and 4 inches, and of 100 feet to 2,000 for the 3 inches, 2 inches and 1 inch pipe.

If the space to be devoted to the experiments be curtailed to the extent of necessitating that the pipes be thrown into folds by semicircular curves, the consideration will arise as to whether the number of folds or elbows in the longer pipe should be in ratio to length and number of elbows in the shorter of the two pipes, or whether there being only one or two elbows in the latter, there should be but the same number in the former, to render the comparison both theoretically and practically acceptable.

There is a great future for McGill in this matter of hydraulics, or of water in motion, the statics of the subject being better known. As the writer has already said, he hails with satisfaction this taking up of the subject of "water in motion" under the different circumstances of comparative lengths, diameters and number of bends or elbows, and of what degree or radius of curvature in ratio to length of curve, and he looks forward to the day when McGill, with its splendid and complete plant adaptable to the study of all such questions, may further enlighten Canada and the world at large on this, as yet, not sufficiently known subject of hydraulics; or at any rate of that branch of it dealing with the flow of water through pipes.

Corroborative of the result arrived at by the author in 1885, in his experiments on the Quebec Water Works system; he may remind the profession that some few years ago he was engaged with Messrs. Keating, Kennedy, McConnell and Beaudry on a matter relating to the Sherbrooke system, where it had been shown that while, during a term of more than eight years, the diameter of pipes had been considered ample, having up to that time given full satisfaction under domestic requirements, they were found to be entirely deficient and useless when, at the fire of the National Bank on Wellington street, and though at the lowest level of the city, the flow of water through some 1,500 feet of 4 inch pipe was so retarded by friction that the five jets only reached to the lower portion of the third floor of the building, which, being four stories high, with high roof or attics, the whole block of buildings containing the bank premises, was reduced to ashes.

The writer was also lately engaged with Mr. Gallagher, for many years Water Works Engineer and Manager at Quebec, and with Mr. Gauvin, an engineer of the Department of Public Works, P.Q., on the question of the adjudication of prizes ($1,000.00 and $500.00) to the competitors for the proposed water works, drainage and electric lighting for the town of Levis (South Quebec), and that one of the salient and pertinent points for consideration was the comparative quantities or lengths of the smaller size pipes, as relating to fire protection. It was this among other pertinent matters that was instrumental in securing the first prize to Mr. R. Surtees, C.E., of Ottawa, he having in accordance with the present practice of the profession, almost totally discarded the 4 inch main, except for short stretches of a few hundred feet, while the other competitors had indulged, as heretofore, in the use of thousands upon thousands of these small bore pipes, so inefficient, when in long stretches, in the dealing with a fire or conflagration of any extent; a case of false and mistaken economy.

And as again further corroborative of the importance of distinguishing between the requirements of a merely domestic water supply, and one where fire requirements are to be considered, the writer would remind the profession that in mostly every other case where his services have been required in relation to the efficiency or the contrary of small bore conduits, he has invariably found that, while small diameters of 4 and even down to 3 and 2 inch pipes are sufficient for family supply or domestic purposes, they and even those of 4 inches are absolutely inadequate and altogether useless when fire protection is to be considered, which, of course, it always should be, since the matter of fires and fire insurance premiums, as well in the merest town or village, is proportionably just as pertinent as in a large and crowded city or community; the last case with which the writer is now concerned, or on which he has been

Mr. T. Drummond.

retained, being that of the thriving parish of Riviere du Loup, or Frazerville, where an aqueduct has just been put in, while doubts as to its efficiency are already cropping up.

Referring to the discussion upon this paper, various questions have been asked. The reason that such large discharges are recorded for the ½ by 2, 1 by 2, 2 by 2 and the 1 by 1 inch orifices is that these small orifices run full like a short tube when the orifice is 2 inches thick. Partial contraction probably occurs, but the jet issuing from the orifice at the outer edge fills the whole orifice there, and the discharge and the coefficient are both largely increased. The ½ by 2 inch orifice taken as the standard of the unit Miner's inch in the Mining Act of British Columbia, runs full as above in the horizontal and vertical positions at heads of from 6 to 12 inches, and it also actually runs 2.147 cubic feet per minute at a head of 8 inches above the centre of the orifices, while it is tabulated to run only 1.68 cubic feet per minute, simply because it runs full as above. The 1 by 2 inch orifice contracts when in the vertical position, but when arranged in the horizontal position it will sometimes contract and sometimes run full, and by rubbing the inside of the orifice it may be changed from one condition to the other. The 2 by 2 inch orifice runs partly full, that is to say, the lower half of the issuing vein where it curves downward fills about half of the orifice, while the upper half contracts. This condition of flow increases both the discharge and the coefficient of discharge, the flow becomes irregular, and it is therefore impossible to measure a single Miner's inch. For the above reasons one-twelfth of the flow from the orifice 6 inches wide and 2 inches high was chosen as the unit Miner's inch. The above orifices under higher heads would, doubtless, contract, and the probability is that the 3 inch wide and 2 inch high orifice would also contract at the low heads. The meaning of the terms "steady stream" and "vein runs full," has also been asked. The first simply means that the issuing stream contracts and runs in a steady, regular manner. The second term refers to the phenomenon described that the orifice runs full. The term is not quite correct, and should only be applied to the phenomenon known as the inversion of the vein, where a rectangular orifice, vertical or horizontal, will give a jet which is horizontal for the vertical orifice, or vertical for the horizontal orifice, and it also develops wings and rays. All orifices except circular ones, or those nearly circular, present this phenomenon, and it has been described in the paper. It is not so marked, under low heads though many of the orifices experimented upon showed it well. My experience of the West has taught me that the value of the Miner's inch usually adopted is 1½ cubic

feet per minute, and it is also a convenient number for dividing by in making reductions from cubic feet to Miner's inches, and I have, therefore, retained this value. For the orifice 2 inches high the head which most nearly approaches this discharge per minute is 6¼ inches above the centre of the orifice. The 6 by 2 inch orifice gives a value of 1.4982 cubic feet per minute, which increases as the orifice widens with the same height and the mean value of the Miner's inch from the various widths is practically 1½ cubic feet per minute. The flow from the orifice 4 inches high is less in proportion to the area, and the probability is that a head of 6½ inches would approach nearer the flow of 1½ cubic feet per minute than 6¼ inches. The same value has been assumed throughout, and values for the different orifices up to 100 Miner's inches has been tabulated and shown. For larger discharges co-efficients would have to be taken to cover the conditions as nearly as possible, and the discharge calculated. As stated in the paper, it is very difficult to deliver exact quantities of water. One hundred Miner's inches covers the requirements for all drifting mines—in fact, for all mines except the larger hydraulic mines—and the determination up to 100 Miner's inches will be found accurate. The value of the Miner's inch varies in the different mining centres, instead of being the same as it should. Endeavours will be made to have the same values throughout the Dominion. The paper has been changed somewhat since the advance proof was issued, and the author thinks its value has been considerably increased. Additional work has been done, and the Miner's inch is now tabulated and given up to and including 100 Miner's inches, and the paper should be of great service in the delivery of water. It must be borne in mind, however, that the conditions of the edges of an orifice has a great effect upon the discharge. They must be square, sharp, and right-angled, and the more they vary from this the greater will be the discharge.

## TO THE THREE OTHER LOW-LYING WARDS.

JACQUES-CARTIER, ST. ROCH AND ST. PETER.

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Arago—Aqueduct..... | 86 | 400 | through Arago Street and Valier St. to | | | | | | | | | | | |
| 22 | Notre-D. St.—Ch. M'ket. | 5 | 481 | 130 | 2,150 | 4,800 | 600 | 3,440 | .... | 11,120 | 7 | 25 | 58 | 30 | To 2nd story of 5 story bdg. |
| 23 | Peter—James ..... | 18 | 468 | 25 | 2,150 | 3,575 | 600 | 3.440 | .... | 9,790 | 6 | 24 | 56 | 35 | To 3rd story Quebec Bank. |
| 24 | Charles—Paul ..... | 15 | 471 | 15 | 1,800 | 2,050 | 600 | 3,440 | .... | 7,905 | 4 | 35 | 81 | 45 | To eaves of Lemesurier Blc. |
| 25 | Paul—Nicholas ..... | 17 | 469 | 15 | 800 | 2,050 | 600 | 3,440 | .... | 6,905 | 4 | 70 | 163 | 80 | To flag staff of highest bdg. |
| 26 | Richardson—Dorchester. | 26 | 460 | 615 | ..... | ..... | 1,275 | 3,440 | .... | 5.330 | 4 | 96 | 223 | 60 | Over all 2 and 3 story bdgs. |
| 27 | Richardson—St. Ours... | 26 | 460 | 1,666 | ..... | ..... | 1,275 | 3,440 | .... | 6.381 | 4 | 95 | 220 | 30 | Over one and 2 story bdgs. |
| 28 | Crown—Prince-Edouard. | 27 | 459 | 1,165 | ..... | 175 | 1,575 | 3,440 | ... | 6,355 | 4 | 86 | 200 | 30 | Half height of Pion Factory |
| 29 | Dorchester—Prince Ed... | 26 | 460 | 715 | ..... | 175 | 1,575 | 3,440 | .... | 5,905 | 4 | 89 | 207 | 30 | 3rd floor Barbeau block |
| 30 | Dorchester—Jac.-Cartier | 10 | 476 | 1,315 | 575 | 700 | 1,575 | 3,440 | .... | 7,605 | 5 | 100 | 233 | 20 | 2nd floor of woolen factory |
| 31 | Dorchester—Panet.. .... | 20 | 466 | 15 | 575 | 700 | 1,575 | 3,440 | .... | 6,305 | 5 | 100 | 233 | 60 | To eaves of Marine Hospt. |
| 32 | Crown—Prince-Edouard. | 23 | 463 | 15 | ..... | 175 | 1,575 | 3,440 | .... | 5,205 | 4 | 95 | 221 | 80 | Over N. S. R. highest bdg. |
| 33 | Bridge—Prince-Edouard. | 17 | 469 | 25 | ..... | 2,175 | 600 | 3,440 | .... | 6,240 | 6 | 87 | 202 | 80 | Over 3 story bdgs. |
| 34 | Queen—Dominique. .... | 16 | 470 | 740 | ..... | 1,850 | 600 | 3,440 | .. | 6,630 | 4 | 80 | 186 | 35 | Over 2 story bdgs. |
| 35 | Desfossés—St. Roch.... | 17 | 469 | 25 | ..... | 2,075 | 600 | 3,440 | .... | 6.140 | 5 | 85 | 198 | 90 | Over 3 story bdgs. |
| 36 | Bridge—Valier.... ..... | 39 | 447 | 15 | ..... | 1,625 | ..... | 3.440 | .... | 5,680 | 5 | 80 | 186 | 80 | Over 3 story bdgs |
| 37 | Valier—Chapelle ..... | 51 | 435 | 15 | 750 | .... | ..... | 3,440 | .... | 4,205 | 2 | 90 | 209 | 70 | Over highest bdgs. |
| 38 | Caron—Valier .. . .... | 50 | 436 | 20 | ..... | ..... | ..... | 3,440 | .... | 3.460 | 2 | 90 | 209 | 90 | Over 3 story bdgs. |
| 39 | Crown—Joseph ..... | 9 | 467 | 25 | ..... | ..... | 825 | 3,440 | .... | 4,290 | 4 | 105 | 244 | 100 | To flag staff Laliberté |
| 40 | Dorchester—Charest.. .. | 26 | 460 | 15 | 500 | ..... | 350 | 3,440 | .... | 4,305 | 4 | 104 | 242 | 100 | To flag staff Bresse factory |
| 41 | Joseph—Caron..... | 26 | 460 | 565 | 800 | ..... | 350 | 3,440 | .... | 5,155 | 5 | 90 | 209 | 70 | To spire Eglise de la Congr. |
| 42 | Charest—Anselme .·.... | 26 | 460 | 265 | ..... | ..... | 1,550 | 3,440 | .... | 5,255 | 5 | 100 | 233 | 50 | To roof Migner factory. |
| 43 | St. Ours—Valier..... | 48 | 438 | 40 | ..... | ..... | 1,900 | 3,440 | .... | 5,380 | 3 | 99 | 230 | 90 | Over roof of Botterell fact. |
| 44 | St. Ours—Arago. ..... | 67 | 419 | 915 | ..... | ..... | 1,650 | 3,440 | .... | 6,005 | 7 | 90 | 209 | 50 | Over 2 story bdgs. |
| 45 | Valier—Voltigeurs.. .... | 48 | 438 | 15 | ..... | ..... | 625 | 3,440 | .... | 4,080 | 3 | 100 | 233 | 80 | Over 3 story bdgs. |

[Signed] C. BAILLAIRGE,

*City Engr. & Engr. New W. W.*

## STATISTICAL DATA—QUEBEC WATERWORKS, 1889.

**To the Four Upper Wards and One of the Low Lying Wards. St. Lewis, Montcalm, Palace, St. John and Champlain.**

| No. | Gauged 14th to 17th August, 1885, between 10 a.m. and 2 p.m. — Locality, Street or Place Intersection of. | Height above high Water level of s ring tides. t. Lawrence. | Head below lip of dam at Lorette. | Through the undermentioned lengths of pipe of vary ng size, from Mount-Pleasant 245 ft. above base, 251 ft. below head at Lorette, to the four upper wards a d Champlain ward, the remotest lower, off the 30″ main 40,800 ft long. From Aqueduct St. 18″ old main, 39,200 ft. through Arago St. 14″ pipe to th three other low lying wards. — Ft. of 4″ pipe | Ft of 6″ pipe | Ft. of 8″ pipe | Ft. of 12″ pipe | Ft. of 14″ pipe | Ft. of 18″ pipe | T. tal length in ft. | No. of angles & sharp b nds. | Gauge pressure in lbs. | Straight height due to pressure. | Approx. height of jet through 50′ 3″ canvas hose and 1 1/4 nozzle. | Building or object, over, against, or near which, the jet was sprung. — REMARKS. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | St. Denis—De Brebœuf..... | 256 | 230 | 185 | 720 | 1,370 | ..... | ..... | 5,130 | 7,405 | 9 | 33 | 77 | 50 | Eaves of highest buildings. |
| 2 | Parlor—St. Louis..... .... | 187 | 299 | 15 | ..... | 1,870 | ..... | ..... | 5,130 | 7,015 | 6 | 64 | 149 | 90 | Over chimney Music Hall. |
| 3 | Ann St. at bend..... | 160 | 226 | 400 | 525 | 2,045 | ..... | ..... | 5,130 | 8,100 | 8 | ... | .... | 50 | Ursuline block, over roof. |
| 4 | Louis—D'auteuil..... | 214 | 272 | 30 | ..... | 1,095 | ..... | ..... | 5,130 | 6,255 | 7 | 63 | 146 | 80 | Over 3 story Ursuline block. |
| 5 | Eustache—Grande Allée.... | 251 | 235 | 15 | 3 | ..... | ..... | ..... | 5,130 | 5,148 | 4 | 73 | 170 | 100 | Over Parl. Bldgs. towers. |
| 6 | Lachevrotiere—Grande Allée | 306 | 180 | 15 | ..... | 30 | ..... | .... | 4,305 | 4,350 | 4 | 50 | 116 | 70 | Over highest buildings. |
| 7 | Scott—Grande Allée..... | 324 | 162 | 45 | ..... | ..... | ..... | .... | 3,100 | 3,145 | 4 | 46 | 107 | 55 | Over dome Bilodeau's block. |
| 8 | Salabery—Grand Allée..... | 301 | 185 | 1770 | ..... | ..... | ..... | ..... | ..... | 1,770 | 3 | 60 | 138 | 70 | Over St. Bridget asylum. |
| 9 | John—Salabery..... ..... | 235 | 251 | 9 | ..... | ..... | ..... | ..... | ..... | 9 | 3 | 91 | 212 | 20 | At Griffith's on 4 in. rider. |
| 10 | John—Sutherland..... | 214 | 272 | 30 | ..... | ..... | ..... | 1,225 | ..... | 1,255 | 3 | 80 | 176 | 85 | Over highest buildings. |
| 11 | Claire—Richmond ..... | 124 | 362 | 25 | .... | ..... | ..... | 2,730 | ..... | 2,755 | 4 | 104 | 242 | 90 | Over highest buildings. |
| 12 | St. Patrick—Berthelot..... | 130 | 256 | ..... | 130 | 360 | ..... | 2,300 | ..... | 2,790 | 4 | 75 | 174 | 80 | Over fire station et al. |
| 13 | John—Augustin..... | 136 | 350 | 15 | ..... | 25 | ..... | 3,115 | ..... | 3,155 | 3 | 74 | 172 | 80 | Over highest buildings. |
| 14 | Eustache—Olivier..... | 97 | 389 | 625 | ..... | ..... | ..... | 3,600 | ..... | 4,225 | 6 | 93 | 216 | 120 | Over 6 St. Grey Nuns' conv. |
| 15 | McMahon—Union..... | 98 | 388 | 675 | .. | ..... | ..... | 4,665 | ..... | 5,340 | 4 | ... | .... | 120 | Over School and Sp. Church. |
| 16 | Hebert—Famille..... | 97 | 389 | 15 | 10 | 750 | ..... | 5,465 | ..... | 6,240 | 3 | 72 | 167 | 80 | Over chim. School of Med. |
| 17 | Mountain—P. Dauphin..... | 129 | 357 | 40 | ..... | ..... | 675 | 6,075 | ..... | 6,790 | 2 | 76 | 176 | 85 | Over Bishop's Palace. |
| 18 | Champlain—Fire Station.... | 29 | 457 | 12 | 650 | 1,400 | 3.730 | 6,075 | ..... | 11.867 | 3 | 70 | 164 | 70 | Over 3 story building. |
| 19 | Champlain—at Lampsons... | 6 | 480 | 12 | 1,650 | 1,400 | 3,730 | 6,075 | ..... | 12,867 | 3 | 57 | 132 | 50 | Over roof all 2 story bdgs. |
| 20 | Champlain—at Cap Blanc... | 6 | 480 | 152 | 2,960 | 1,400 | 3,730 | 6,075 | ..... | 14,317 | 3 | 55 | 128 | 25 | Over roof all 2 story bdgs. |
| 21 | Champlain—Cul-de-Sac..... | 27 | 459 | 14 | ..... | ..... | 2,025 | 6,075 | ..... | 8,114 | 3 | 120 | 279 | 150 | Over highest bdgs. Duf. Ter. |

Mr. W R Butler.

The experiments recorded in the paper are full of interest, and they have evidently been carried out by the author with much care and industry.

The results seem to agree, in the main, with others which have been published bearing upon the effects of variation in the values of the ratios between head, width, height, and length, upon the coefficient of discharge; the values of peculiarities of these coefficients as tabulated may no doubt be accepted for application where the conditions are similar to those under which the experiments have been conducted, and where $H_1 . H_2$, and $H_1, -H_2$, are all small quantities.

The existence of critical values of the above ratios, governing the suppression of contraction, illustrates the danger which sometimes exists of making too general an application of results obtained experimentally under special conditions; their existence is a matter of some moment to the buyers and sellers of water under the module referred to.

The British Columbia definition of the "Miner's Inch," had it stopped short at the first part and insisted also upon the sharp edges and free overfall, would probably not have been very wide of the mark; how much more than the stipulated 1.68 cubic feet of water customers have been obtaining for their "Miner's Inch" has often no doubt been dependent upon the condition and shape of the walls of the orifice (frequently perhaps roughly and carelessly shaped) and possibly orifices may be found in operation producing all sorts of variations between complete and suppressed contraction. It occurs to the writer to question whether the orifice 2 inches square, which is suggested, with the attendant other relative dimensions, might not in some cases be inconvenient, where less or more than the four "Miner's Inches" was to be delivered, or where, in irrigation works, the necessary position of the orifice made it difficult to fulfill the other required conditions.

Might not the quantity be gauged, in almost all cases, by means of a weir or open notch—which is probably the most satisfactory form of gauge for outdoor situations, the box or reservoir suggested, in such case, being made to discharge over the weir, and communication between the box or reservoir and the main flume, to be by means of orifices of the shapes which have been commonly used, but made capable of adjustment, as to their dimensions, by means of a sluice gate or shutter. Any required head might in this way be maintained over the sill of the weir, and the quantity might be adjusted with a very fair approach to accuracy.

The "Miner's Inch" might still be defined as 1.68 cubic feet of water per minute—or say five-thirds of a cubic foot, and the method perhaps would involve little change in the existing order of things and might legalize the measurement of the water upon a principle sufficiently accurate for all practical purposes.

Thursday, 6th December.

G. H. DUGGAN, Vice-President, in the Chair.

The following donation to the Library was reported:—
"Some Details of Water Works Construction," from Mr. J. D. Mackerras, A.M.Can.Soc.C.E.

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# ABSTRACT.

## PAINTS AND VARNISHES.

By A. H. SABIN.

(Journal Assoc'n. of Engineering Societies, Feb., 1900.)

Cement is strongly alkaline, and as alkalies do not attack iron, the iron will be preserved as long as the alkaline quality persists; but cement is somewhat porous, and the protective effect may be less if the iron is submerged, or in the presence of acid gases.

Asphalt is non-porous, and insoluble, and an excellent covering in situations where a layer of considerable thickness is used, but it will melt if subjected to heat, and is not self-supporting.

Paints furnish a non-porous and durable protecting film. Ordinary oil paints consist of linseed oil, and some pigment, which is added to harden and thicken the film, and to fill up the pores in the oil film.

The pigment red lead is entitled to a place by itself. It is a powerfully basic substance, which combines chemically with the oil, forming an insoluble, hard, tenacious mass. Chemically it is known that commercially pure red lead, composed of nothing but lead and oxygen, is a varying mixture of protoxide and peroxide of lead, the latter being from 45 to 90 per cent. The correct proportions to make the best paint are not known, and possibly much of the uncertainty attaching to the use of red lead is due to this.

Linseed oil possesses the property of thickening rapidly on exposure to the air, in a higher degree than any other oil. This is not due to evaporation, for it absorbs and combines with the

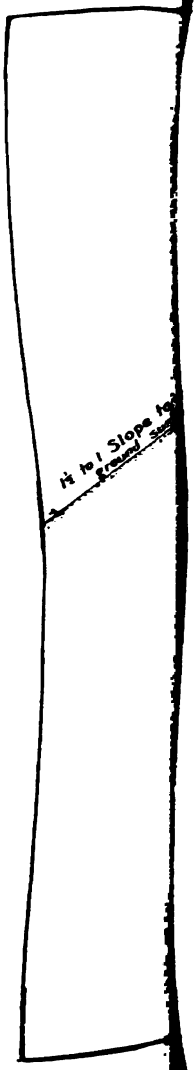
oxygen of the air, increasing in weight. A paint made with it may be handled in five of six days. "Boiled" oil is raw linseed oil treated so that it will dry in twenty-four hours. Many processes are used, and therefore it appears that, while raw oil is a definite substance, boiled oil is a name applied to a variety of manufactured products. It is possible to make it from fish oil, or heavy mineral oils, which are much cheaper than linseed oil.

These latter practices of systematic adulteration are especially followed by the manufacturers of paints. They take the ground that the users of paints ought to pay a fair and legitimate price for linseed oil paints, and that if they prefer to purchase at a lower price, it is the business of the paint manufacturer to supply at a reasonable profit a paint that will meet their requirements. It requires a tedious and expensive analysis to detect these adulterations.

Varnishes consist of linseed oil mixed at a high temperature with a vegetable resin. When properly mixed, the kettle is allowed to cool somewhat, and sufficient spirit of turpentine is added to make the varnish flow properly when cold. Varnish can be applied in a thicker film than oil, makes a harder film, and is less porous. If properly made and proportioned, it is more valuable than an oil paint. As a matter of fact, it has always been used over paint to protect it from the weather. The question naturally arises, why not further increase the permanence of varnish by the use of a pigment, as with oil. Pigments do increase the durability of varnishes, and by far the most permanent paints which it is possible to make are made in this way. These will not be so brilliant and lustrous as the varnishes alone, but more so than the oil paints, and smoother and harder. To remain sufficiently elastic and tenacious, they must be properly made, the commonest adulterant of varnish being rosin, a comparatively small amount of which will completely ruin the durability of the best varnish.

The objection to the use of varnish paints is their cost. Suppose it costs twice as much as an oil paint, which would probably be a minimum. The minimum cost of applying any paint will be as much as the price of the oil paint. Then when applied the costs will be as two is to three. If the varnish paint will last 50 per cent. longer than the oil paint, it will be as economical. It is the opinion of the writer that it has been proved by actual use to be more than 100, and in many cases 300 per cent. more lasting.

The comparative value of the different paints and protections used, is difficult to determine. The writer is of opinion that there is no very decided difference in the efficiencies of the various pigments, that varnishes and varnish paints properly made are better than oil paints, that the latter give better average results than red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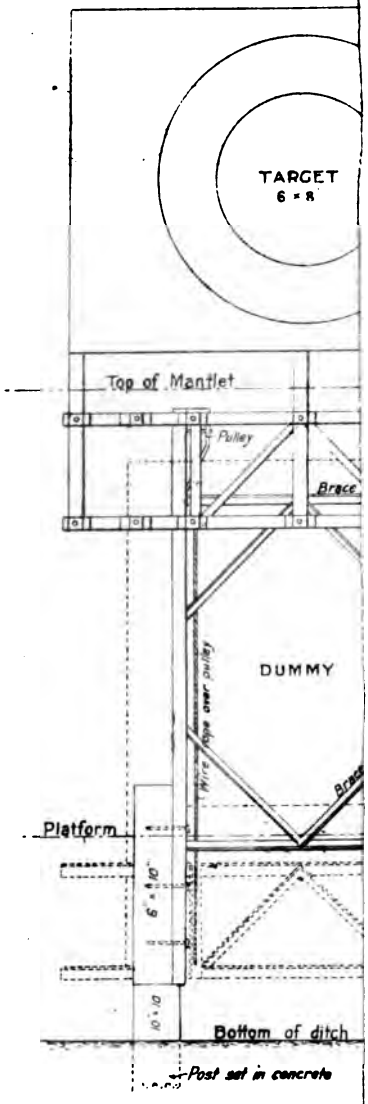


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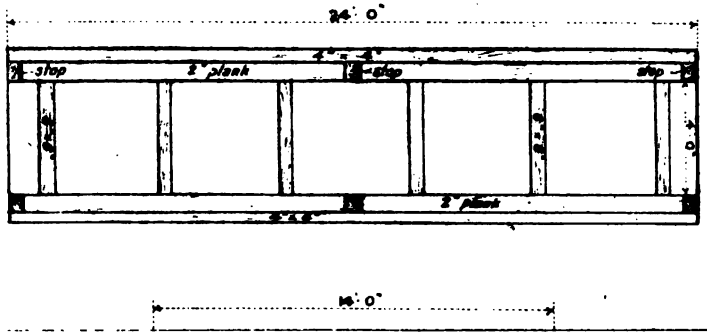


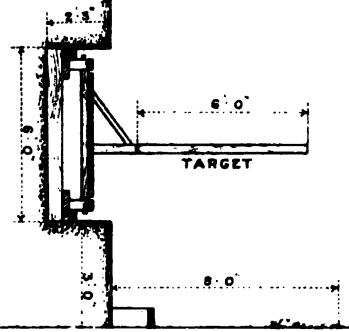


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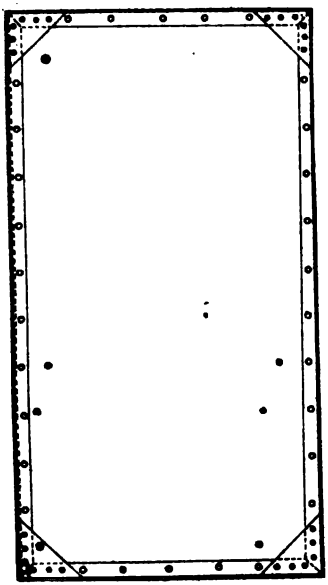

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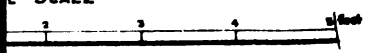


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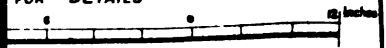

lead, but that the best possible results with red lead are considerably better for most exposures than oil paints, and that adulteration is a common and serious evil.

This latter is the fault of the purchaser rather than the maker. If the former would not buy an inferior article, the latter would not make it. The writer knows cases where paints, the materials to make which, as specified, would cost $1.30 a gallon, were supplied, guaranteed to pass inspection, at 60 cents. The purchaser is to blame for this, in letting contracts at prices which, if the contracts were literally carried out, would entail the ruin of the contractors. These men are the real builders of the great engineering works, and are entitled to fair pay for their labour and skill, and when they are told that they will not be paid for a good article, they are likely to supply an inferior one. Since the range of prices for paints for structural work is about 400 per cent., it is natural that inferior articles should be often substituted for better ones in this line, if they are in any.

J. G. K.

Thursday, 20th December.

G H. DUGGAN, Vice-President, in the Chair.

***Paper No. 155.***

## NOTES ON CANADIAN MILITARY RIFLE RANGES.

BY G. S. MAUNSELL, A.M.CAN.SOC.C.E.

There are in Canada about one hundred military rifle ranges registered in the Department of Militia and Defence, varying in number of targets from one to fifty-five. Ranges of one and two targets are used by individual companies. From eight up by a battalion or more.

Before the issue of the Lee-Enfield rifle, the old Snider, with which the Canadian Militia was armed, was comparatively easily provided with a safe range. With the modern arm, and as the country becomes more thickly settled, it is a much more difficult matter.

No rifle range is absolutely safe, particularly where inexperienced men are firing. There is always the richochet or wild shot to guard against, to say nothing of the rifle being discharged prematurely.

The extreme range of the Lee-Enfield bullet is between 3,500 and 4,000 yards. The following thicknesses of materials are proof against bullets from this rifle:—24 inches of earth, not rammed, ⅜ inch mild steel plate, 38 inches fir, and 24 inches oak.

The question of background behind targets is, therefore, a very important one. If the country beyond is level (even with an artificial butt), the length of ground behind the targets should be at least two thousand yards. If a natural stop butt or hill is provided, the slope behind the targets should be one over one, to prevent richochets, and should be fifty feet in rear of targets. If the proper slope is not naturally provided, it should be scarped.

With reference to the construction of markers' mantlets, stop butts and targets, the Department of Militia and Defence has adopted a standard which is carried out by the Engineer Branch, under the supervision of the Chief Engineer. This standard, so far, is applicable only to the larger ranges, and is adopted on the Rockliffe Range, Ottawa, and at Hamilton. Others are now in construction under this system.

This standard (see Plate 10) consists of a trench for protection of markers, about four feet deep, according to the nature of the ground,

with a retaining wall in front eight feet high above markers' platform, and natural slope of earth in rear. A stop butt, twenty-five feet high on level ground, is placed fifty feet in rear of the trench, constructed of crib work with stones. The target frame (see Plate 11) used in this trench was devised by the Chief Engineer, Militia and Defence, and consists of a steel frame holding any size target; the target sliding up and down on pulleys, counterbalanced by a second target or dummy. The dummy is used for marking. If two targets are used on each frame, discs are used for marking.

The construction of the different ranges, other than the standards, varies in different districts, according to the nature of the ground, number of targets required, etc.

When the nature of the soil permits, and drainage is obtainable, a trench seven feet deep is dug for the markers, and an ordinary "Window Frame" or revolving target is used.

If the nature of the ground is rocky, and not easily excavated, a crib work mantlet (see Plate 12), eight feet high over markers' platform, is erected, filled with earth, eight feet thick at top, and lined on interior face with ⅜ inch mild steel plates. From this mantlet target carriages are run out on tracks on either side, with targets which are run in to patch and mark.

A standard Morris Tube portable rifle range (see Plate 13) has also been adopted by the Department of Militia and Defence, and constructed by the Engineer Branch. This consists of a portable steel mantlet for the protection of markers, with two miniature targets each, one on either side. It is used either in drill halls, armouries, or at camps of instruction. A Morris Tube is placed in the barrel of the Lee-Enfield rifle, and a miniature target used at a range of about 25 yards. All the Canadian transports for South Africa were fitted with these ranges; two for rifle practice (four targets), and two for revolvers (four targets).

A system of movable targets is now being devised by the Engineer Branch, Militia and Defence, for the larger ranges. This consists of a double line of rails at reverse slopes, so that the carriage, containing target, runs down one and back the other. This carriage may contain a target of a running deer, disappearing man, or any such device. Earth works are thrown up to protect the markers and target carriages.

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E. H. Keating

# TRANSACTIONS

OF

# The Canadian Society of Civil Engineers.

VOL. XV., PART I.

JANUARY TO JUNE,
1901.

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## INSTRUCTIONS FOR PREPARING PAPERS, ETC.

In writing papers, or discussions on papers, the use of the first person should be avoided.

They should be legibly written on foolscap paper, on one side only, with a margin on the left side.

Illustrations, when necessary, should be drawn on the dull side of tracing linen to as small a scale as is consistent with distinctness. They should not be more than 10 inches in height, and *in no case* should any one figure exceed this height. Black ink only should be used, and all lines, lettering, etc., must be clear and distinct.

When necessary to illustrate a paper for reading, diagrams must be furnished. These must be bold, distinct and clearly visible in detail for a distance of thirty feet.

Papers which have been read before other Societies, or have been published, cannot be read at meetings of this Society.

All communications must be forwarded to the Secretary of the Society, from whom any further information may be obtained.

The attention of Members is called to By-laws 46 and 47.

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Thursday, 3rd January.

DR. H. T. BOVEY, President, in the Chair.

## CORRESPONDENCE ON RIFLE RANGES.

MR. C. R. COUTLEE thought that the subject of rifle ranges was one well worthy of serious attention by engineers, and one in which they might well assist their country by introducing modern ideas and cheaper and more efficient constructions.

He thought it well said that ranges were never safe with the extreme killing distance of modern rifles. The greatest danger was, of course, behind the targets, but accidental shots might carry laterally, and in general the residents near a range were in a state of suspense while matches were in progress. This rendered shooting somewhat unpopular and the ranges few, limiting the free use of the soldier's only tool. Even where ranges were provided, men were required for marking, etc., with consequent expense and trouble to the volunteer, so, practically, ranges were used by but a few enthusiasts near towns.

He thought that this state of affairs could not be compared with the inducements to proficiency in shooting that would follow the establishment of numerous local ranges provided with self-registering targets and covered-in firing butts.

As to the requirements of such ranges, the area of land should be as small as possible to limit expropriation expenses. The length should be, roughly, a mile, so means should be taken to allow of as narrow a strip as possible, consistent with efficiency. A range located lengthwise of a valley or along the base of a steep hill would at once secure side protection.

The surface between firing point and target should be graded level to admit of the fire plane being as close to the ground as possible. It was very important to inculcate the habit of shooting low, in fact, it would be well to count no hits above the top of the bull's eye.

The firing points were generally earth mounds at fixed distances, 200, 500, etc., yards from the targets. The contestants, scorers, etc., moved all their belongings at each change of range. It would be preferable to have a fixed covered-in firing point, and targets arranged to approach and recede. A graded range would allow of operating targets on light railways. Such targets might be self-registering or consist of steel cabins mounted on wheels, the marker

being inside and having telephone connection with the firing point Moving targets were very serviceable in giving experience in the judging of distances. He thought that at the end of the range, where at present an earth "parado" or stop butt was used, it would seem better to erect a steel plate stop screen, say 25 feet high, supported by angle frames and securely riveted. It would cost from $5 to $10 per linear foot and afford greater protection to the public both from high shots (the targets being much lower than at present) and from ricochets. Steel screens would also allow of operating disappearing targets in several ways. Cable ways, stretched between target cars, above described, were also worth experimenting upon. Stop screens might possibly be constructed of bullet-proof cloth held between 1 inch board sheeting. A general fault of the standard target shown was its height above ground, which encouraged high shooting, while hitting a man lying down, or running in a stooping posture or just appearing over a ridge, always close to the ground, would seem to be the practical requirement. The retaining wall shown seemed to be too deeply founded for a standard section. About 3 ft. below the surface would suffice, and all the wall should be concrete if it paid to use that material in any part. There was no frost batter, and no stone filling was shown in rear of wall to allow drainage down to the weep holes. The elaborate concrete slab between the marker's platform and the stop butt did not appear necessary. The author had drawn attention to setting posts of target frames in concrete, which only enlarged the diameter of the post and would not prevent settlement. This difficulty sometimes occurred with posts from which car gauges were hung or from which gear connections were made, and resetting became difficult. Concrete foundations should be horizontal, not vertical. The Morris Tube shields appeared to be a thoroughly good and well thought out design. Good marksmen were produced on the prairies because ranges were everywhere, and constant use of the eyes at long distances improved the sight. He considered that numerous rifle ranges, planned as indicated, and administered somewhat after the style of the familiar "shooting gallery," would surely tend to diminish our lamentable inefficiency in musketry.

MR. W. R. BUTLER remarked that though it was but to repeat what was insisted upon in the paper, it might be reiterated that too much care could not be exercised in the securing of ample range behind the target. It should certainly be not less than the 2,000 yards specified. The question was whether even this was sufficient. An earthen stop butt, whether natural or artificial, would need to be of very considerable height and extent to afford anything like adequate protection, and unless the material should be of the nature of pure sand it could hardly be considered a preventative of ricochets.

With recruits, the wildest shooting, and all the risks of accidental discharge, would have to be provided for, and, with reference to the design of mantlet shewn—it might perhaps be questioned whether the target *frame* were sufficiently below the top of mantlet to guard against splintering of the nickle cased bullet, in a "dropping shot," upon any iron work in the frame. It was not an uncommon thing for the protecting roof of the mantlet to be struck by a bullet. The plate did not show distinctly of what material this roof was formed, and the details of target frame for side targets were not shewn. Referring to the masonry retaining wall, he thought the question would arise whether, with some soils, the top part of the back of the wall was protected from damage by action of frost. This part of the wall would need to be kept very perfectly intact, otherwise the steel plate might perhaps become as welcome an addition to the markers as it must be in the case of the crib-work mantlet. It had been reported to him that various plans had been proposed for ranges, where a great deal of shooting was done, having for their object the collection of the metal of the bullets, etc., and that at Bisley, for instance, £100 per annum was easily cleared over and above the cost of digging and collecting. Being a civilian he could not presume to criticise the paper, and had submitted what had been suggested by an examination of the diagrams in the light of what limited opportunity he might have had of noticing the incidents of range firing, either directly or indirectly.

Mr. Macpherson thought that it would appear, at first glance, that the subject of Military Rifle Ranges had but little to do with Civil Engineering, yet there were engineering problems involved, and not only engineers, but public spirited men of all classes should take an interest in everything which would tend towards making every able-bodied man in the country, if not an expert, at least a fair rifle shot. He thought that our recent experience in South Africa should teach us a lesson as to the almost inestimable defensive value of the modern rifle in the hands of men trained to use it; but as Canadians were peaceful people, not likely to be called upon to fight in any great aggressive war, the great value of the rifle, as a weapon of defence, should be particularly reassuring to us. The great majority could only learn how to shoot at rifle ranges, hence their interest to engineers became apparent, and the Author of the notes on ranges had shown clearly how they were constructed. It remained for us to criticize and suggest improvements, which was rather a difficult matter for a civilian to attempt, even though he may have had the benefit of military training in youth, as such rapid strides had lately been made in modern arms and all that pertained thereto. He noted that a system of moveable targets was being devised by the Militia Department, and he would venture to suggest

that they should go furtner and discard fixed targets entirely. In actual warfare, or any kind of shooting to kill, the rifleman had to judge his own distances. It would immensely improve his ability to judge these distances correctly, if all firing on ranges were done at moveable targets at uncertain distances. A system of targets with protection for markers could be devised so that the whole device could be moved together and used at any rate and all distances. The Morris Tube ranges, mentioned in the notes, were a valuable adjunct to real ranges, and, in the absence of the latter, were invaluable, especially in the preliminary stage of marksmanship.

MR. J. W. HARKOM commented upon the unsuitability of the present standard target on account of its height, it being too costly to construct in rocky ground and hard to drain in low ground. It was also complicated, and likely to get out of order. He expressed approval of Morris tubes and of Coutlee's system of movable ranges, the latter especially for training and practical shooting. The introduction of long range rifles, he stated, had rendered the possibility of securing ranges much more difficult than formerly, on account of the danger from wild shots and ricochets. He approved of the use of permanent shooting ranges with targets in echelon, and of the use of screens to protect from wild shots. He believed that shooting galleries would take the place of rifle ranges in the vicinity of large cities, being supplemented by an occasional day's practice in the country. Except in very favourable locations, shields should be used on all ranges to deflect wild bullets to the ground, and the ground of the ranges should be soft.

MR. R. W. LEONARD observed that Mr. Maunsell had given the thickness of various materials which were proof against bullets from the Enfield rifle. A little more definite information as to the distance such bullets would penetrate these materials would be of interest, as he had seen it stated, by a credible authority, that at a distance of thirty feet these bullets would pierce six inches of fine dry sand confined between one inch boards.

The AUTHOR, replying to the correspondence, wished to point out to Mr. J. W. Harkom that the standard target frame could be used at any depth (the bottom of the target being either on a level with the surface of the ground or above it, according to circumstances), but would require a certain depth of trench. The construction of this trench would be also necessary for the protection to markers. The depth, as was pointed out in the "Notes," would vary with the nature of the ground. If the nature of the ground were such that a trench was not practicable, it was pointed out that other forms of construction were adopted, such as the lateral targets shown on plate 12. The standard target frame had not been found at all complicated, nor likely to get out of order if properly set and balanced. It was very necessary that targets be placed as near the ground as

circumstances would permit. The standard target frame allowed of this, with the usual trench required for all target frames. The standard frame took up the minimum lateral space, thereby reducing the width of the range, which was an important point in considering the cost where a large number of targets were provided. This was not the case with the target shown on plate 12, or with revolving targets, where much more lateral space was required. The construction of rifle ranges with targets in echelon and one firing point for all ranges had been found very satisfactory at Toronto, but was a much more expensive method of construction, and, therefore, not generally adopted. He would say, with reference to Morris Tube practice, that since his article was written, Morris Tubes had been practically done away with, and a cartridge of full size, but with a reduced charge, had been adopted with great success for gallery practice. The same marker's mantlet was, of course, used with this ammunition as with the Morris Tube. Mr. Coutlee's idea of covered-in firing points was hardly necessary for practical work, nor would it tend to teach the soldiers active service conditions. A graded range, to allow of operating targets on light railways, would be a most expensive arrangement where a large number of targets were required, but might be feasible on a small scale. The self-registering target was a good suggestion, and had been tried, but, as yet, without very satisfactory results. The difficulty was to get accurate marking. He did not consider a steel plate stop butt practicable for large ranges. Although the cost of the plates only for a butt of ⅜″ steel 25 feet high would probably be about $10.00 per lineal foot, the additional cost to properly stay this shield would be very great. In any case it would be questionable if this structure would withstand the great wind pressure. The concrete between the marker's platform and stop butt kept the target posts dry and the trench from being washed out. A cross section of the target posts only was shown in the drawing accompanying the notes, whereas a longitudinal section would show the foundation of concrete between each pair of posts horizontally, as suggested, with longitudinal cedar feet joining each pair.

The Author, in reply to Mr. W. R. Butler, remarked that the steel target frame was six inches below the top of the retaining wall or marker's mantlet when in the firing position. This space was considered and had been found ample. The trajectory of the Lee Enfield was very flat. The bullet had an angle of descent of 1 in 22.5 fired at 1.000 yards. The "protecting roof," referred to by Mr. Butler in the marker's shelter in Plate 10, was only for the purpose of protecting the marker from the sun or rain, and was not required for protection from bullets; nor could this roof be struck by a bullet passing over the wall. It was constructed of a wooden frame,

shingled. The detail of the construction of targets for the lateral target carriages in Plate 12 was practically similar to those for the steel frames, the legs of the wooden target fitting into slots in the frame on the target carriage. There was no necessity for a steel plate in rear of the masonry retaining wall, as, even without the earth protection in front of this wall, there would be 2′ 3″ of masonry or concrete to stop the bullet, but the earth was necessary to prevent a ricochet. In the case of the cribwork mantlet the steel plate was used only as a precaution against a dropping shot cutting through the interior upper angle of the crib. With reference to bullet catcners, suggested by Mr. Butler for all rifle ranges, bullet catchers were used in Canada with fair results with the Morris tube or miniature ammunition ranges, but the expense of providing them for large ranges would be very great. On either ranges the amount of lead obtained from the small bullet now used did not warrant the expenditure involved in their construction.

He gives the following list of materials, with the approximate penetration of the Enfield bullet in each at 40 yards range, as asked for by Mr. R. W. Leonard:—

Clay, 3-6 feet, according to its character.
Earth, free from stones, (unrammed) 22 inches.
NOTE.—Ramming earth reduces its resisting power.
Sand, 18 inches.
Soft wood, 36 inches.
Hard wood, 22 inches.
Wrought iron, ⅜ inch.
Hardened steel plate, 3.16 inch.

MR. P. WEATHERBE remarked that both standard and miniature ranges were being equipped throughout the Dominion by the Canadian Government almost as quickly as suitable sites were discovered in localities where they would be used. The notes under discussion described the construction of these ranges in their various forms, as carried out in this country up to the present time. Several points had been raised in discussing the subject which would be touched upon here. Miniature rifle ranges or galleries could never take the place of standard ranges, as the conditions were entirely different with reference to light, wind, condition of the atmosphere, etc. Experience had shown that men might be excellent rifle shots in these galleries who did very poor shooting when on a full sized range. On the miniature range a man could be taught how to aim, how to hold his rifle with the sights vertical and the consequences of his not doing so, etc., but it would still be a long time before he became a practical shot in the field. Miniature ranges should, therefore, be used for preliminary practice and to stimulate the interest

taken in rifle shooting. Moving and stationary targets were both excellent, but the latter should be used until a man became a fairly good shot before any useful practice could be obtained with the former. Moving targets of various kinds certainly stimulated the interest taken in rifle shooting, and should be installed wherever possible. Experience in Canada showed that wherever rifle ranges were provided the interest taken in rifle practice was great. When the Lee Enfield rifle was introduced there were so few rifle ranges in Canada which were considered safe for practice with this rifle that this interest had dropped to a considerable extent; in fact, nearly all the Government ranges had to be closed at that time until they were rebuilt or repaired and made safe. This work was now under way and almost completed except where the site itself was found to be unsafe, when the range was closed. The large expense necessary to purchase safe sites and construct safe ranges in thickly settled country naturally limited the number that could be equipped each year. The interest in rifle ranges, therefore, could not be increased as much as desired until a sufficient number of ranges were provided. The expense to the rifle shot was at present very little where ranges were provided. Rifles and free ammunition were furnished by the Government to all military and civilian rifle clubs under certain conditions, and the expense for markers, etc., was practically nothing. Three men could practice on one target at a time, and markers could be obtained for from 15 to 25 cents an afternoon. Self-registering targets for a standard range were difficult to obtain and expensive to construct; few, if any, that were practical were on the market. The standard targets and frame that had been adopted by the Militia Department were examined and reported on very favourably by a committee of the Dominion Rifle Association and officers of the Imperial and Canadian Military Service. They had given satisfaction almost without exception wherever used during the last four years, from British Columbia to Nova Scotia. The length of a range for the Lee Enfield rifle depended to a great extent upon the conformation of the ground. If on a plain it should be about two miles to ensure safety, unless a very high stop butt were constructed. The extreme range of the Lee Enfield rifle had been found to be 3,760 yards when all conditions were favourable. To guard against lateral ricochets the range site should increase in width from the firing point in the direction of the targets. A range in a valley with high steep ground in rear of the targets lessened the danger, but it was difficult to obtain such a site. It was not necessary to level or grade the site for a range, although it was desirable. It could seldom be done at a low cost. An uneven site fulfilled more the conditions of active service, and could be used to advantage to show men how cover could be obtained in the field. A steel plate stop butt would be expensive and would increase

the danger of ricochets and splashing back of bullets. Many of the new nickel-coated bullets, striking in the same place would pierce a steel plate, unless of an impracticable, and costly thickness. The marker's trench, as at present constructed, was placed as far below the natural ground surface as possible, consistent with good drainage, and gave excellent results. Targets representing men were used in military matches, and might be introduced more generally. The retaining wall, shown in the sketch, must have its foundation below the frost line, or the result would be disastrous. The construction followed for the last four years, was, if anything, too solid. There was a severe lateral strain, at the surface of the ground, on the target posts. It had been found necessary, therefore, to set the posts in concrete to prevent them becoming loose in the ground. There was no necessity for any extensive foundation for frames, as their weight was comparatively little.

**Paper No. 156.**

# MASONRY DAMS AND RETAINING WALLS IN GENERAL, CONCRETE WORKS, ETC.

By Chas. Baillairge M.A., F.R.S.C., M.Can.Soc. C.E.

It must be admitted that "retaining walls" in Canada have had but a poor or short-lived record; unless it be argued that their durability should, in the interests of the working classes, not be too persistent. But, as an engineer, the writer cannot admit this. He has been associated with the reconstruction of many of these walls, and has witnessed the failure of so many others as to be entitled to say a word on the subject.

The City of Quebec is pre-eminently one of retaining walls; many of its streets on the inclined or hilly frontage of the town having had to be supported on the down side to level them up to necessary grade.

These walls have almost without exception been made unduly thin to withstand the original pressures against them, or the stresses liable to obtain thereafter.

The Des Carrieres Street wall of some 600 feet in extent, in rear of Dufferin Terrace, erected about 60 years ago, under Government control, stood its own for only 30 years, and was rebuilt again under Government superintendence and at Government expense, it being then an enclosure wall of the so-called Governor's Garden of the period. This second edition of the structure again gave way to the earth pressure in rear of it, and was rebuilt or erected a third time within the last few years under the writer's term of office as City Engineer, and at the cost of the City.

The retaining wall of the old parliament site at the head of Mountain Hill has during the same period of 60 years been rebuilt not less than twice.

The same fate has overtaken the Artillery Street wall, the Dambourges Street wall; and most of those along the north side of Cote d'Abraham, and other streets, have had to be rebuilt during the writer's time, and generally after a persistence of only 10 to 20 years or less.

The splendid cut stone retaining wall along Commercial Street, Montreal, though built hardly 50 years ago, is and has been for some years back giving way, in like manner, from earth pressure

in the rear; and the walls of the new Louise Docks at Quebec have already become disgracefully out of plumb and alignment from the same cause.

The impression, of course, has been in all these cases, or was at the time of construction, that the several walls mentioned, having nothing to withstand or bear up against but the pressure of so much dry earth behind them, they needed not to be of a thickness or breadth of base greater than from one-fifth to one-third their height, sometimes even less than this, and as low as nine-seventh, or about 3 ft. for a 20 ft. wall.

The fact was overlooked that the dry earth backing might become waterlogged by infiltration, and especially so where no weepers had been provided for the removal of wet and moisture; or even with weepers, if the filling in their proximity was of too retentive or impermeable a nature, or where, in the case of weepers or no weepers, there had not been interposed between the back filling and the rear face of the wall a narrow space filled with loose material of a stony consistence to allow water, when reaching the wall, to run to the bottom thereof and thus find its way out or expend itself by absorption into the underlying soil.

The writer has noticed in very many cases of such retaining walls and of those separating tenements on sloping ground, or lying back to back (the one at a higher, the other at a lower level; the ground embanked on the one or upper side to raise it to level of street on that side ; excavated on the lower side) that the percolation into the embankment had absolutely waterlogged the filling, and thus caused it to press with the pressure of a liquid or fluid on the intervening wall, causing it to bulge or swell, and thus fail by disintegration, or give way bodily.

Not that this waterlogging of the filling was of a permanent nature, nor need it be; but only of momentary duration, as it might produce a thrust forward of the wall, of only the fraction of an inch, thus enlarging its (the water's) containing space, and making room for itself to subside in, and gradually pass through the wall to the lower level, or evaporate or be absorbed by the subsoil. This waterlogging would, of course, be intermittent, and might recur again at every succeeding return of heavy rains, and the wall again be thrust forward, and each time by an additional fraction of an inch, and up to final destruction or overthrow of the stony structure.

An unmistakeable proof of this intermittent and instantaneous action of water, and of the fact of its thus acting during only a second, or fraction of a second, is still to be seen along the rear face of Dufferin Terrace, Quebec, where, in an illustrated paper read before this Society in the early nineties, and published in

its Transactions, it was shown by the writer that an open fissure in the cliff in rear of the terrace had, under a persistent and heavy rain of several days' duration, in September, 1889, become filled with water, when, due to the thrust from the rear, an outward portion of the cliff fell forward, causing the destruction of much property and the loss of some 52 lives. The crevasse alluded to, thus filled with water to a depth of over one hundred feet, exercising a hydrostatic pressure which pushed the intervening rock forward by some six inches.

Could this pressure have been continuous, the thrust of the cliff forward would also have followed suit, or been continuous, the aperture widening more and more till the whole cliff fell forward; but it was not continuous, nor could it be, for the moment the fissure became enlarged the water level fell, and the pressure ceasing, the motion forward stopped short at the same time.

In addition to this action of a fluid or waterlogged substance against a retaining wall, there is also, in this climate, the yearly action of frost, which, in expanding the back filling, as water does in freezing, also pushes a wall forward or towards the open, and though but as little as the eighth or even the sixteenth part of an inch at a time. results in course of years in its overthrow.

It is therefore seen that all retaining walls are liable to become dam walls, which have to stand the pressure of water, and that as such the same rules as to strength and thickness should apply.

Also that where ice may or might form in rear of a wall, and where this ice in forming cannot spend its effort on compressible material towards the rear, as where such substance is solid rock or equivalent thereto, the whole effort is then against the wall, and, if repeated from year to year, the wall must go, as nothing can withstand or nullify the effect of ice expansion; since, as is well known, if an iron shell be filled with water, hermetically plugged, and then exposed to frost, the shell, however thick it may be, will burst. In such a case the only safeguard for the retaining wall is to so roof in or impermeably cover the surface of the back filling with asphalt, or other watertight substance, that no water can get at it, or penetrate the soil in rear of the wall in a way to run the risk of any ice forming, as said above.

It may also be remarked here that a Mr. Bone, engineer of the Oregonia Bridge Co., has recently patented and built at Columbus a concrete-steel retaining wall, a section of which is illustrated on page 448 of the N. Y. "Engineering Record" for November 10, 1900. The thickness of this wall at base is less than one-sixth its height, and a simple glance at the figure will show, without resorting to any calculation of strains or resistances, that, unless the back-filling be self-supporting, such a wall can not continue for any length

of time to stand the pressure of any ordinary back-filling against it without it tending to bend over or give way. The stiffening ribs of the iron or steel screen behind the wall are not nearly deep enough to act as efficient buttresses. They should for this purpose reach much further into the backing. The rod tying the base of screen to the foundation can have no effect in preventing it from tilting forward. For this purpose the tie rod should have been attached to screen at say two-thirds of its height, and reached out far back to some unyielding pile or concrete anchorage.

The subject of retaining walls would not be complete without an allusion to wooden walls instead of masonry for the same purposes. The author in his time has had occasion to build and rebuild at Quebec, thousands of feet lineal of street supporting structures *et al*, of this kind, along the St. Lawrence and St. Charles water frontages, and also along the side hills of the city. The respective lives of these walls may be taken at from 15 to 20 years average for spruce, and 25 to 30 years for pine timber; while the portions under water, and even those extending to a few feet above low water, may endure indefinitely as far as material is concerned; but these walls, even though secured by cross ties running or tailing deep into the back filling, or to a depth, for the upper ones, fully equal to height of wall or wharf, give way in course of time from the earth pressure and frost, and thus gradually lose their batter (generally of 1 to 1½ in 12) by about a quarter or half an inch per annum, becoming vertical, and then begin to over-plumb. About the time they are decayed to an extent to require repairing or rebuilding, the over-plumb reaches such a figure that they are ready to topple forward—that is, their fronts or timbered facings, which become torn away from the ties that head into them, as the outer ends of these ties or portions exposed to wind, water and weather also decay, thus allowing the front timbers to be thrust forward, whilst such portions of the ties as are buried in the back filling continue to endure and can outlive the outer timbers for years thereafter.

On the rebuilding of the facings, short ties may be thus used by spiking them to the remaining portions of the old ties, and when on a slope or side hill with tendency to slip forward, it is a good and even necessary precaution to bolt and, better still, to fox-bolt the ties to the adjoining or nearest rock, or attach them to piles or suitable pickets driven deeply in to the subsoil; and this precaution is none the less pertinent in dealing with stone walls, which may thus be made to endure for a longer term of years.

Of course, if the back filling is of stone, more or less stratified and piled in a way not to have a tendency to thrust forward, or if the filling is of such a clayey nature that percolations from the

surface may, instead of sinking into it, run off towards the open and find their way through or between the face timbers, the wall may then endure during its natural life as regards decay without losing its plumb or perpendicularity.

These facts are coming to be appreciated, and it is satisfactory to state here that with our engineers in general (we notice it most in the works of the C. P. R., the Grand Trunk, and others of the kind where retaining walls are visible) extra thickness now prevails, and has for some years past, and precautionary measures are resorted to rendering all such work more permanent than heretofore.

The recent giving way of a coffer dam at the Boston Navy yard is also a case in point, where the filling in was similar to a fluid substance, and required a thickness of wall equal to that of one having water to bear up against.

These failures are the best, the most instructive lessons to an engineer when the cause of failure can be got at or made known. Nothing is to be learned from a structure of any kind which holds its own or persists, and does not give way, except that it is evidently strong enough; but it may also be too strong, ten times, a hundred times too strong, as of a column or pillar, or pier, or wall, which could bear ten times, a hundred times the weight it is called on to support. Now, the coffer dam alluded to was on sloping ground, with a consequently varying pressure of water against its sides, and the dam therefore made thicker, or the clay or puddle enclosing space between its walls wider at one point than at another, the dam being some 6 feet in breadth at its highest or shallowest portion, or inland, while increasing to not less than 36 feet at a salient angle where the depth of water rendered it pertinent to do so. The contractors were under the idea probably that the clay filling was or would be of such a nature as while watertight or impermeable, to stand its own, and be self-supporting, as would be the successive courses of masonry of a stone dam, each underlying course supporting the one above, and all those above it without crushing or giving way by spreading, like a plastic substance under a superincumbent weight. Under this idea of the non-giving nature of the clay, and of its self-supporting nature, the walls enclosing this dam were built of superimposed timbers of only 8 inches square at bottom, and thinned out to 4 inch timbers at top, and to prevent the inbursting action of the water from without, these walls were braced with 8″ x 8″ timbers running horizontally across the dam from inner to outer wall thereof. The outer wall or face towards the sea was, moreover, braced against giving way outwardly by sloped struts abutting against the bottom or ground of the foreshore, there being no danger of the structure giving on the in-shore side, where the ground was high and sufficiently resisting to prevent any tendency of bursting outwards on that side.

The horizontal ties above alluded to (some 36 feet in length) do not even seem to have been supported as they should have been against sagging at the centre, as the engineer in charge tells us that the weight of clay as dumped into the dam enclosure caused these ties to sag and break; when, under the plastic and spreading nature of the filling, the outer wall for some 50 feet of its length was thrust forward and the clay filling ran out of it towards the sea.

This example is here introduced as proof of how the most compact filling material when dumped through water and becoming intermixed therewith. may reduce to a semi-fluid substance, and exert the same pressure against its retaining walls as would so much water.

This coffer dam is now being reinforced, as it should have been from the first, by a structure of solid wooden crib work built outside of it, and of proper breadth of base with stone filling to prevent the spreading of the dam walls, and thus, as the writer holds, this failure of such a dam and in such a depth of water as 30 feet must prove to be a most instructive lesson to every engineer or contractor in going over the same ground.

Let us again here repeat the maxim that there can be nothing so instructive to the architectural or engineering profession as a failure of any kind when its why or wherefore can be arrived at, and the writer takes this opportunity of suggesting to the members of the respective professions, engineers, contractors and overseers, the advisability whenever it can be done, in the case of any accident to a dam or other structure of whatever nature, of immediately, or before the debris are cleared away, enquiring into the cause or causes of such accident, and of reporting them for the information of professionals in the respective lines.

## STONE OR MASONRY DAMS.

The author has for years past contended, and sees no reason as yet to be of a different opinion, that stone dams should at every point of their height be at least as thick as the height or depth of water above that point, including overflow, which means in other words, that the weight of masonry of the whole dam or above a horizontal plane at any portion of its height—stone masonry being assumed as of twice the weight of water—should be twice that of the pressure or weight of water above such level. This, of course, refers to a dam built in horizontal courses of masonry.

If the virtue or binding qualities of the cementing material could persist for all time, or for so many years as the dam were expected or called on to endure, this thickness of dam would not be advocated; but that it does not persist beyond an indefinite number

of years, was shown some few years ago by the failure of the Bouzey Dam in France, when some 232 lives were lost by the rush of water through the gap in the dam; and that it may not even persist for a single year, by the failure of the Austin Dam, so-called, or, in fact, that, as in that case, the cementing material between the courses was possessed of no adhesive quality whatever in the way of making a monolith of the whole structure.

Then, if the cement can not be relied on to bind the structure into an indestructible whole, nothing remains to stand the pressure of the water tending to push the dam forward, but the weight of the structure and its co-efficient of friction of stone on stone or masonry on masonry. This co-efficient is known to be but .5, though given as .7 to start a stone from a state of repose, which it is evidently not prudent to rely on, as shown by the accidents alluded to at Bouzey, Austin and elsewhere. Portions of each of these dams were moved bodily forward or down stream, without being either overthrown or disintegrated. An endeavour has been made to show that the toe of the Austin Dam was undermined, but this only corroborates the writer's views, as proved by the fact that when there was no more resistance at the toe to stand the thrust, the only remaining opposition to the dam moving forward was its adhesion to its foundation, and its weight. And yet, in spite of such presumable adhesion, and in spite of its frictional resistance, it was thrust completely forward, and to a considerable distance down stream.

What other conclusions, then, can be arrived at than that it was deficient in weight and therefore in thickness to such an extent as not even to develop an amount of friction capable of causing it to hold its own, or to resist the tendency of the water to push it forward, which it must have done had its weight under a co-efficient of 0.5 been at least twice that of the impounded water, or pressure due to weight or depth of water.

The conclusion to be arrived at is, therefore, inevitable, the author takes it, that for horizontally coursed masonry, where the cementing material has ceased to be binding, and where nothing is left to resist its being thrust forward but its co-efficient of friction 0.5, its weight should be twice that of the total pressure tending to move it forward, or, in other words, it should be (taking its weight as already set forth at twice the weight of the water, or say 125 pounds to the cubic foot) of a breadth of base at bottom level of reservoir or of impounded water equal to the height or depth not only of the water so impounded, but of the upper surface of the water when over-topping the dam—by whatever number of feet so over-topping it (11 to 12 feet in the case of the failure at Austin) or with an overflow of that number of feet above the crest of dam.

The theoretical section of dam would thus be that of a triangle, or in the case of no overflow the crest of dam would be a mere edge or arris, or of a width at top equal to the height of overflow, but in practice this upper edge is thickened out for practical considerations to at least 3 or 4 feet, to allow of heavy and resisting coping stones, and as a pathway for foot passengers across the dam, while it may and generally is extended out to such a number of feet (8 to 10 feet) as will allow of a wheeled vehicle passing over it with room on one side to meet a foot passenger on the way, and this extra thickness at top is brought about by gradually curving out and up the down face of the dam in a way to give the required width at top.

A dam when intended to answer as a pathway or road from one bank of a river to the other, should, of course, be provided with a spill-way or freshet water pass, though, in the case of the 12 ft. depth of overflow at the Austin, no reasonable size of spill-way could have been devised, no system of pipes and gates from the bottom which could have even commenced to deal with such an excess of water, and as such an eventuality may happen in the future, which may not have happened for a quarter or even half a century, as at Galveston, it is better to be on the safe side and in proportioning the dam allow for a possible overflow of such a number of feet as circumstances may render possible even if not probable; since the difference in cost can thereby be only slightly enhanced, which is absolutely as nothing beside the terrible disaster of loss of life and of having to expend hundreds of thousands of dollars to rebuild a dam, and during an interval of months or even years leaving a whole town or village or a city in total darkness and without power to drive its electric cars, or to work any of its public or private machinery or industries.

The foundation of a dam should of course be dug down to a solid and impermeable substratum, and this, stepped up and down to allow of the masonry or concrete interlocking in a way that there may never arise the eventuality of such a structure sliding forwards on its base or bottom.

What precedes, as already stated, relates only to horizontally coursed masonry dams, and where material is at hand to give the dam its required weight and thickness; but when material is scarce or expensive, the want or deficiency can be made up for or compensated in either of two ways, to wit: by throwing the courses up from a horizontal position on the up dam side to an inclined one on the down side, say by tilting up the beds or courses in a way to have them normal or perpendicular, or more or less so, to the outer slope or apron of the dam. By this system a much lesser thickness of dam wall can be made to answer; the tendency of

the dam to go forward being in such case resisted not only by the force or weight necessary to overcome the friction of stone on stone, or of masonry on masonry, but, in addition to that, the force required in pushing the structure forward to raise it along an inclined plane and under favorable circumstances; a dam thus constructed might possibly be made of only half or two-thirds the thickness advocated for horizontally coursed masonry.

The other mode of constructing, to avoid thickness, is or would be that of the Eddistone Lighthouse in the British Channel between France and England, that is, by dovetailing and interlocking all the component stones of the structure, and that, of course, not only horizontally, but also vertically, and the lower foundation courses thus also vertically into recesses cut purposely in the solid rock, or again, may be, by fox-bolting every course, the one to the other, and the whole to the rock foundation.

This salient feature in the erection of masonry dams is, the author is glad to notice, coming to be recognized, and his advocacy of breadth of base equal to height is taking hold of the engineering mind, as at Massachusetts, Jersey City, Denver, etc. The dam of our Quebec Impounding Reservoir at Lorette or St. Ambroise, and stretching for 200 feet in length across the river St. Charles, was built by Mr. G. Baldwin of Boston, a very safe and cautious engineer, and moreover fifty years ago, of a thickness not only equal to the height or depth of water impounded, but, as if by intuition, to such a further thickness or breadth of base as to allow for an overflow of some three feet; and on more than one occasion since 1851, when the dam was put in, the overflow has quite or nearly reached this figure, though ordinary spring freshets cause the water to overtop the crest of dam by only two feet or less, and even this does not happen every year, the normal overflow varying from an inch or two to eight or ten.

## CONCRETE DAMS.

The profession has now taken to the building of dams of concrete instead of masonry. We now have in the Province of Quebec at least some five dams so constructed; one at Chambly, one at Lachine, one across the Jacques Cartier River, one across the Montmorency, and one just completed across the Chaudiere, a few miles west of Quebec. Mr. Stearns, of the Chaudiere works, is a specialist at concrete structures, having built entirely of concrete an arched bridge of some 30 feet span, and other structures. This gentleman is of opinion that concrete dams, on account of their thus monolithic nature, are, and will prove to be stronger or more resisting than dams of stone masonry, and what is of great importance, that they can be built for half the cost of dams of masonry, which would

seem to be the case when it is considered that the dam across the Chaudiere river, which is 850 feet in length, 23 feet broad at base, 20 feet 3 inches high above bed of river, and thus as thick as it is high, costs but $200,000.00, with all the adjuncts of a spill-way, an enclosure of double the height for the three penstocks to draw from, each of them over 8 feet in diameter, running out and down the face of cliff as at Niagara, in a way to give a head of 110 feet at the turbines, the cost of which, and of the power house and rock excavated tail race are all included in the above sum of less than a quarter of a million of dollars, while intended, as they are, to supply Levis, St. Romuald and adjoining parishes with electric light and power plant to drive electric cars.

It has just been said that the Chaudiere dam carries out the author's theory of breadth of base equal to height or depth of water impounded, and allowing for overflow, while at Lachine and Chambly, at Jacques Cartier and Montmorency, the breadth of base is fully equal to and even in excess of depth water held back.

Referring again to the Massachusetts dam over the Nashut at Clinton, 35 miles from Boston, which is to impound 63,000 million gallons of water (the greatest reservoir in the world after that of Perigar in India, and only less in height than the great Crotan dam now under construction for the New York Water System), its length being about 1,400 feet, and height at centre 200 feet; this dam, which at first sight appears of less thickness than that advocated by the writer, is in reality found to be of such equality, when allowing for its great thickness at top, and its height, 20 feet above level of water in dam; that is, it gives the requisite weight duly to resist any tendency to thrust it forward, and, moreover, two of its features are those already alluded to, it being specified by Mr. Stearns that courses of masonry shall rise toward the downward face of the dam by one in four (1 in 4), while the contract also requires that the making of horizontal beds of masonry shall be avoided, but that on the contrary the beds shall be stepped, and that in each underlying bed there shall be stones projecting in a way to become incorporated in the next course above.

The greatest precautions have been taken in this case to guard against failure on account of the numerous and populous communities below the dam, and the important manufacturing interests centering there—for instance, some 875 borings have been made to an average depth of 20 feet, aggregating 16,905 feet lin.—the rock has been drilled to reveal its nature or geological features, as well as its position, in 38 places, giving together a depth of 2,489 feet, one hole being as much as 286 feet deep.

The contractors, as a prudential feature, are to stop short with the blasting before they reach the actual depth at which the

masonry is to be started, and the remainder of the rock thence removed by barring and wedging, so as not to disturb the underlying beds or strata in a way which, by causing them to become loose, might possibly be creative of leaks beneath the dam. The prospecting has been more thorough here than ever before in works of like nature. This drilling led to some heretofore unexpected features in the strata, such as finding beds or pockets of gravel beneath the solid rock or granite.

The same thing, however, was met with in the case of the dry dock at Levis, opposite Quebec City. Under a stratum of hard pan there was found a bed of sand of considerable thickness, such that the entrance to the dam had to be moved in, and the dam thus shortened by some 65 ft., requiring an expenditure of $118,000 for a coffer dam—all this and the curtailment due to not drilling down or boring some distance into the apparent rock bottom, which would have revealed the fact, and saved the large expenditure. This graving dock is now being lengthened from the rear end, and, in fact, now-a-days harbors are being dug, as at New York at present, to a depth of 40 feet, in view of the ocean liners of the future drawing 37½ ft. of water (the difference, of course, to allow for "swell" of ocean), while docks are being made 1,000 feet in length, preparatory to that length of vessel being obtained early during the forthcoming century, to commence on the first of January next (1901).

Another extensive dam now under construction, as mentioned in the New York "Engineering Record" of October 13th, page 349, is that for the Denver W. W. It is being built across the so-called "Goose Creek Gorge"; its height of impounded water is to be some 200 feet. This dam, it is true, is but 168 feet base, but the upper base or summit has a breadth of not less than 28 feet, increasing the base of the theoretical triangular section to 196 feet, which is nearly the depth of water to be impounded, while the additional height of dam more than makes up the difference in weight required to stand the thrust.

A 3,400 feet dam is also being built some 70 feet high at Boonton for the Jersey City Water Works. In writing to a New York engineering paper on this subject of dams, the author says, and the same thing may be repeated here:—"Is it not "strange that, while we are so particular in recommending "a factor of safety of from 3 to 5 in bridges and other structural "iron work, etc., and of as much as from 5 to 10 in structures "where disintegration is possible under heavy loads and the effects "of weather, wear and tear, we have been in the habit of constantly "neglecting any factor of safety at all, so to say, in the erection "of such structures as dams, where failure must and almost always "does lead to dire loss of life and property."

Nor need it be said here that Mr. Stearns has no doubt adopted the far more prudent system of not only giving the dam its proper cross section of weight equal to more than double that of the water impounded, as here advocated, but that in requiring in addition to that, that the beds be tilted, and the courses made purposely non-continuous and stepped, while introducing stones vertically binding the courses, the one to the other, as an additional precaution against any portion of the dam having a tendency to slip forward over the courses below, he is doing nothing more than introducing, for the first time perhaps, a proper factor of surety, a prudential and really essential one.

The power works above alluded to, have all developed within the last five years, and so has the "St. Lawrence Power Works" at Massena, where the dam has an extent of over 700 feet. These works are remarkable as outstripping those at Niagara, where, under a head of 150 feet and turbines mounted on vertical shafts, the power developed to date, and though comparatively colossal, is but 50,000 H. P, while at Massena, instead of ten 5,000 H. P. turbines, or sets of turbines and Westinghouse generators, the number thereof is to be 15, giving 75,000 H.P. with a capacity in the canal or head race of double that figure, or 150,000 H. P. Massena, it may be said, is on the American side, and at the head of the "Long Sault" rapids, the canal or head race being some 16,200 feet in length, with 6,000,000 cubic yards of excavation, now all but completed. The fall is 50 feet, and available head 40 feet, but the supply of water much greater than that which, from the same source, has been made available at Niagara, since the whole flow of the St. Lawrence or drainage from the upper lakes runs through the Niagara river to Lake Ontario, the delivery over the falls being 18,000,000 cubic feet per minute. To utilize the available head the turbines at Massena are mounted on horizontal shafts, and deliver into a small river, the Grasse, which thus answers as a tail-race for the outflow—though it still remains to be seen whether, as with the so-called drainage works at Chicago, there may not arise international difficulties incident on one nation robbing the other of so much water, which, instead of being returned to the St. Lawrence direct, goes off on a side issue; in one case from Chicago to the Gulf of Mexico; in the other, from Massena, as already stated to the Grasse river at a distance of 3 miles or more from the St. Lawrence, while such action may also be fruitful of local complications by flooding the Illinois on the one hand and the Grasse river on the other.

But with this question we have not here to deal, and it is only introduced as a reminder to engineers that such river diversions, even if only partial, may give rise to suits at law, when the engineer

may be taken to task for not having foreseen the trouble and advised his patrons thereof.

## CURVED OR BOWED DAMS.

There is another class of dams where thickness is or may be partly compensated for by bending the dam up stream into a bow on the principle of a lock-gate of a canal. Such a dam occurs at Parsons, Kansas, where a masonry dam of 100 feet reach has been made convex up stream and to a radius of 200 feet. This dam is but 10 feet high, with a base of 6 feet; but as it is 2 feet thick at top, if a line be drawn through its vertical section from its outer edge at base to its inner edge at top, and the triangle thus cut off turned upside down and applied, thus increasing the base by 2 feet, the base will then be within 20 per cent. of being equal to the height, and the arched feature may be taken to make up in resisting strength for the want of sufficient base to conform to the writer's theory of "base of dam equal to height or depth of water, including overflow, if any."

As to this practice of arching a dam horizontally or in plan, Mr. Mansergh, President of the Institution of Civil Engineers, England, in his presidential address of November the 6th, says:—"In 1878 I built a small dam on the River Wyre . . . about 57 feet high above the river, across a narrow gorge, where I could give the wall a curve in plan of only 100 feet radius. In such a situation the structure could obviously be made much lighter than say the Rankin type section for a long straight wall," which corroborates what I have just said of thus adding strength and resistance to a dam wall by throwing it into a bend up stream, and thus saving thickness and weight of masonry.

Mr. Mansergh also strengthens the writer's position on the inadvisability of horizontally coursed masonry in a dam, and the necessity of binding the masonry vertically. In an address he says, alluding to the new Crotan Dam, which is not erected as at first intended, at Quaker's Bridge, but two miles further up the valley: "I have just received some photographs of this work, which " represent the wall as being built in horizontal courses of roughly " squared stones, a method which I do not approve. What we " want in such a wall is ample weight, and good vertical binding, "and this can be secured by building in stones of from two tons to " six tons in weight, practically as they come from the quarry, " and surrounding them with smaller stones and good concrete well " rammed in so as not to leave any interstices."

He then goes on to say, alluding to dams which he is now building across the Elan River: "We are getting, in the Elan walls, " say nearly 50 per cent. of solid blocks, and the whole structure

"will weigh, when complete, 157 to 160 lbs. per cubic foot, and will "practically be monolithic."

And finally, if it be argued that the writer, in assuming stone masonry to be only of double the weight of water, while it is in reality about 20 per cent. heavier, and that he is thus going beyond his own theory of weight of dam equal to twice that of water impounded, he can only urge his theory as rendering assurance doubly sure by thus further increasing the factor of safety.

This subject of dam building is becoming one of vast importance, as all the water powers of the country are being taken hold of for power purposes, or for water works, which are also power problems whereby water may be forced up to a given level without the cost of pumping, and as no one but an engineer can cope with such problems as the harnessing of the rivers of the world to give power to mankind—the truth of the writer's motto, "The Engineer the master spirit of the age," heading an article of his which appeared in the "Canadian Engineer" of July and October last, and reproduced at page 293 of the London "Engineering Times" for September last, is indisputable—for as he truly says in his last paragraph under the above heading:

"There is, however, another and more summary way of judging "of the merits and qualifications of an engineer in comparison to "those of persons exercising other callings. It is this: of an "engineer or almost any other man of common sense, you can "make an alderman, a mayor, a premier, a president, a king, "or a Kruger; and this at, so to say, a moment's notice; but of "none of these can you make an engineer without years of study, "practice and the keenest of observation."

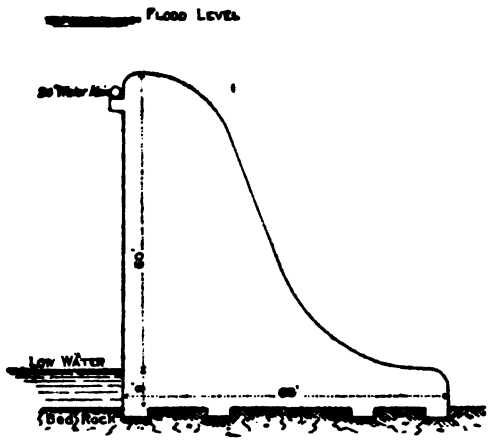
## CORRESPONDENCE.

MR. F. P. STEARNS remarked that the author, in the portion of his paper relating to concrete dams, quoted with a degree of approval the views of another gentleman, "that concrete dams, on account of their monolithic nature, are and will prove to be stronger and more resisting than dams of stone masonry, and, what is of great importance, that they can be built for the half of the cost of dams of masonry." In some situations this statement might be correct, but he cannot endorse the statement as a general one. The Wachusett Dam, over the Nashua River at Clinton, which had been referred to in the paper had been recently let, and the price bid by the successful contractors for the granite rubble stone masonry, of which nearly the whole of the dam was composed, was $3.80 per cubic yard, the masonry to be laid in natural cement mortar mixed in the proportion of 2 parts of sand and 1 part of cement. The same contractor bid $4.35 per cubic yard for concrete masonry of natural cement mixed in the proportion of 1 part of cement, 2 parts of sand and 5 parts of broken stone. The price for concrete would, undoubtedly, be considerably lower if the whole dam were to be built of it, but he did not think the price would be much, if any, lower than that bid for the rubble stone masonry. In this case the granite for the masonry would be obtained from a quarry 1¼ miles from the dam. He cited this case to show that under some circumstances dams of concrete may cost nearly, if not fully, as much as dams of stone masonry. Concrete structures were not as strongly bonded as those built of rubble masonry made of large stones, and when built of considerable length in warm weather had contracted and cracked in cold weather. Such cracking rarely occurred within his experience, in well built rubble masonry. It was claimed that concrete dams, on account of their monolithic nature, had an advantage over stone dams. With regard to this point, it might be said that rubble masonry, when built in a great mass, as in the case of very large dams, was composed of stones so small in proportion to the whole mass that the masonry could be considered as practically monolithic. This was the view taken by Mr. Mansergh with regard to the dams which he was now building across the Elan River, where as stated in the quotation in the latter part of the Author's paper, he remarked that the whole structure "will practically be monolithic"

MR. MOHUN observed that he had lately had occasion to design some concrete sluices in dykes, the walls of which were practically

retaining walls in pairs connected at the base by a concrete invert. The base of the wall reduced to the theoretical triangle equalled four-ninths the height. In this case, owing to the invert, they could not slide on the foundation. The backing was of approved material placed in thin layers, well rammed, and was impermeable. They were further reinforced by collar walls to act as cut-offs, and not with any idea of adding to their stability. He had always understood that drainage was considered essential to the stability of retaining walls for earth unless the backing was of an impermeable character. In the case of Quebec, the Author depicted an unfortunate state of affairs. Ignorant of the locality, he was under the impression that Quebec was a city of terraces, tier above tier, and wondered if it were possible that the lower retaining walls were surcharged. With regard to the thrust due to the expansion of ice, it appeared to him that the frost counld hardly penetrate downwards more than a few feet from the surface, and that the strain would be confined to those few feet at the top of the walls. He thought it possible that the author could state through what thickness of masonry such frost, as was encountered in Quebec, would penetrate and affect the backing, assuming it to be in a semi-fluid condition. He made this remark with some diffidence, having no personal experience in the matter. He also failed to follow the author's deduction that, in the case of the Austin Dam, "the cementing material between the courses was possessed of an adhesive quality," as, surely, if this were correct, instead of huge masses of the dam being torn from thin foundations and hurled bodily down stream, as shewn in the photographs, the dam would have fallen in a heap of debris carried various distances by the current. The result appeared to prove that the weak point there was the connection with the foundation. At the same time, had that been good, it was not proved that the dam had sufficient stability to withstand the enormous additional strain suggested by Mr. Williams, in the "Engineering Record" of the 28th April, who drew attention to experiments made in the Cornell Hydraulic Laboratory, and, as their result, suggested that the pressure at the foot of the dam might have been increased by 40 per cent. owing to a partial vacuum being found behind the falling sheet. This was, at least to him, a new destructive agency to be taken into consideration, though it seemed that air might be easily supplied through pipes discharging it at that part of the face at which a vacuum would otherwise be formed. Taking masonry at 150 lbs. per cubic foot, the approximate weight of the Austin Dam as built, would be about 330,000 lbs. per lineal foot, and by the author's contention it should have been 448,000 lbs., or fully one-third more. It was probable that, with the result in our minds, we should be inclined to agree with him. Again, the "Engineering Record," of the 26th May, in an able article,

pointed out the disastrous results which might follow if the bond with the foundation were deficient, not necessarily throughout, but at any point. The accompanying sketch was an approximate cross section of the Austin Dam, and it might be noted that two of the cross walls shewn on the sketch, but which two the deponent knoweth not, were omitted in construction, and their omission would naturally tend to facilitate the entrance of water beneath the dam.




With Mr. Stearns he is fully in accord as to the substitution of concrete for masonry when practicable, having for many years used it with satisfactory results, and it is believed that it is far easier and less costly to render a concrete structure monolithic than a masonry one. He has also found that first-class concrete weighs from 140 to 143 lbs. per cubic foot, and, in estimating, assumes it at the former. Recent dam failures appear to point to the necessity of making a dam monolithic, not only in its own constituent parts, but also with the foundations and abutments; also waste weirs were to be avoided in dams if possible, otherwise additional precautions should be taken, which would be needless in an impounding wall never overflowed. With regard to the adaptation of the arch to the construction of dams, there seemed to be no reasonable doubt that, if really constructed on the two principles of the arch, a greater resistance would be the result than with straight walls of the same areas of cross section. But to achieve this result it appeared to him that the dam should be absolutely monolithic, or provided with voussois

as in a stone arch, otherwise we might as well expect to obtain increased resistance in an earth dam by making it convex up stream. In his opinion, the author's suggestion as to dams was one calculated to render them safe. As was well pointed out in the paper, the new century would doubtless see many a mountain torrent and stream curbed with these erections, the failure of which would cause incalculable loss of life and money. With regard to retaining walls the case was somewhat different, depending upon varying conditions which should be taken into consideration when designing. He failed to see that any one hard and fast rule could be laid down as applicable to the various classes of backing in different climates, but thought that each case should be considered from the standpoint of the conditions governing it

The Author, in reply, fully concurred in the remarks of Messrs. Stearns and Mohun. He wished, however, to state that he did not intend to give any measure of approval to the assertion of the engineers in charge of the dam at Chaudiere, that concrete cost one half the price of stone masonry. As a matter of fact, he doubted the accuracy of the statement; but, on the other hand, it appeared to him that $3.80 per yard was not an adequate rate for stone masonry, especially when built of such heavy stone as that which should be used for a dam. Referring to Mr. Mohun's question as to the depth to which frost would penetrate, he had seen it in waterwork trenches near Quebec as deep as seven feet. This might not, however, have been due to actual penetration downwards from the surface, but have been induced by currents of cold air following the pipes from their outlet, through loose stone-filling around them. While the water-pressure against a wall must be considered as being more forcible at the centre of pressure, the pressure due to expansion of the backing owing to congelation of the case of a monolithic wall must be considered as acting at near its top, thus causing it to lean over bodily.

Thursday, 17th January.

DR. H. T. BOVEY, President, in the Chair.

Messrs. E. A. Rhys-Roberts, R. M. Hannaford, H. B. Stuart, J. D. Mackerras and A. E. Smaill, having been appointed scrutineers of the ballot for the election of members, reported the following elected:—

MEMBER.

A. W. ROBINSON.

ASSOCIATE MEMBER.

C. H. MORITZ.

ASSOCIATE.

A. CHAUSSÉ.

Transferred from the class of Associate Member to the class of Member:—

A. R. DAVIS.

Transferred from the class of student to the class of Associate Member:—

F. L. JERMYN, W. M. MACPHAIL, J. RANKIN, JR.

STUDENT.

J. SYLVIAN.

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## ABSTRACT.

### THE OZONIZATION OF WATER.

By George A. Soper, Ph.D., New York.

(Abstract from New England W. W. Association Proc.)

Ozonization of water consists briefly in bringing about a thorough mixture of the water to be purified and a quantity of atmospheric air, which has been ozonized.

Ozone is an allotropic form of oxygen (Chemical formula O3). Oxygen in this form posseses great chemical activity, so that, when brought into contact with water containing organic matter, the latter is rapidly oxidized.

The best method of producing ozone is to cause what is called a

silent or brush discharge of an electrical current, of high potential but small volume, to pass across a space containing air, the result being that a small portion of the oxygen of this air is converted into ozone.

The use of ozone in water purification was first suggested by Frölich, an engineer of the German firm of Siemens & Halské, in a paper read in 1891, describing some experiments on the production of ozone in large quantities. He stated that the effect of ozone on impure water was to mineralize the organic impurities, precipitate iron when present, and lastly to render the water completely sterile. Beyond expressing this opinion, he made no effort to establish the usefulness of the process.

The question was then investigated by Chlmüller for the German Imperial Board of Health, with a small experimental plant. He found that he could easily sterilize water rich in bacteria, but only when it contained a small amount of matter in suspension.

The first attempt to utilize the method on a practical scale was made by Baron Henry Tindal, of Amsterdam, with a plant of 7,500 gallons daily capacity, which was built at Oudshoorn, in Holland, in 1895. Water from the Old Rhine, polluted with factory waste, sewage, and farm drainage, was treated; and no difficulty was found in sterilizing it after it had been first filtered. As reported by Dr. Van Ermengem to the Belgian Government, both the high color and unpleasant odor were considerably reduced, and the chemical qualities as indicated by the permanganate figure were improved 50 per cent. Finally, after treatment, this water had a good appearance, was free from taste, and was pronounced to be in all respects excellent for drinking purposes.

During the progress of these experiments the ozonizing apparatus used by Frölich and Ohlmüller, which would only permit of electric potentials of 4,000 volts, was improved so as to be capable of using currents of 50,000 volts. The air to be ozonized was first filtered, then dried and chilled. Sterilization of the water was effected either by spraying or by bubbling. The former gave a more intimate contact, but of shorter duration than the latter, which was also the most economical.

In 1896, at the Hygienic Exhibition in Paris, Baron Tindal installed an experimental plant of 30,000 gallons daily capacity (24 hours). Power was taken from the electric light circuit of the exhibition to operate a small dynamo, whose current was transformed for use 50,000 volts. The same type of apparatus was used as in Oudshoorn. The ozonizers consisted of five glass-lined rectangular boxes 10-in. x 14-in. x 23-in. Suspended from the cover of each were packages of platinum foil sheets, whose edges were separated from the walls of the boxes by a space of about 3-16 inch. On the glass

walls opposite the platinum sheets were fastened thin gold plates. The gold was grounded and the platinum was connected with the electric transformers through glycerine resistance cells placed above the boxes. Air passed through the boxes and became ozonized by contact with the electric flames which played between the platinum and gold.

The sterilizing apparatus consisted of upright cylinders through which water and ozone were pumped, passing from bottom upward, and escaping together at the top.

Filtered Seine water was used. The chemical effects of the treatment were, as reported by Dr. Repin,—chlorine not changed, nitrates reduced slightly, partial precipitation of the carbonates of lime and magnesia and sulphate of lime. Color, odor and taste improved, No bacteriological report.

It was stated that water containing not more organic matter than is represented by 0.4 parts permanganate per 100,000 could be purified with an expenditure of energy equivalent to 1 H.P. per 1,500 gallons (per 24 hours probably).

A third trial plant of 100,000 gallons, daily capacity, was erected by Tindal in 1897, at the Brussels Exhibition. The water from the Brussels filter galleries was easily sterilized, as was also poluted canal waters after having been first filtered. The cost was estimated as about $9.00 per million gallons of water not containing more than ten parts of organic matter per 100,000.

There are at the present time two municipal ozonization plants in Europe, the first of which was built in 1898 at Blankenberg, on the Belgian coast, to sterilize 500,000 gallons per day of canal water, which has to be first filtered.

The essential features of this plant are similar to those just described. The three sterilizers are each 30 feet high and 3 feet in diameter, and consist of glaze-lined flanged iron pipe in 3 feet lengths. At each joint is a screen consisting of a plate of celluloid pierced with small holes, 126 to the square inch, the object being to thoroughly mix water and air. About 2 H.P. is required for the ozonization of the air.

The second municipal plant, and the largest yet built, is situated at Saint Maur-les-Fossés, near Paris. Their capacity is 3,170,000 gallons per day, and the water treated is that of the Marne after being subjected to sand filtration.

The water after treatment is bright and clear, and without odor or taste. Sterilization is almost complete.

The cost of the ozone treatment after filtration is given in some detail as follows:—

| | | |
|---|---|---|
| Energy required for ozonizer.. .. .. .. .. .. .. .. | 3.0 | H.P. |
| " " " air pump.. .. .. .. .. .. .. .. | 1.5 | " |
| " " " water pump.. .. .. .. .. .. .. .. | 12.0 | " |
| Total energy.. .. .. .. .. .. .. .. .. .. .. .. .. .. | 16.5 | H.P. |

| | |
|---|---|
| Cost per day at 2 cts. per H.P. per hour .. .. .. .. .. .. | $7.90 |
| Other daily expenses are:— | |
| Salaries of three men at $200 per month.. .. .. .. .. .. | 6.66 |
| Interest charge of 3½ per cent. upon $20,000 (cost of plant) | 1.94 |
| Interest charge of 4 per cent. upon $20,000 (depreciation) | 2.22 |
| Total.. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. | $10.82 |
| Total cost of ozonizing 3,170,000 gallons of water per 24 hours.. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. | $18.72 |
| Cost per million gallons .. .. .. .. .. .. .. .. .. .. .. .. .. | 5.90 |

Experiments upon the purifying action of ozone alone, and ozone in connection with metallic iron, have been recently made by Dr. Weyl in Germany. The results seem to be due partly to ozonization and partly to coagulation.

Dr. Soper (who has himself made a long series of experiments on the ozone method of purifying water) states that some of his experiments had as much reference to a reduction in color and odor in water as to its sterilization, the germicidal capabilities seeming to have been already proved beyond a doubt. He treated Croton water, and distilled water, in which various colors and odors were produced artificially. With Croton water it was possible to reduce the bacteria 74.8 per cent., the permanganate figure 12.5 per cent., the color 35.1 per cent., and the odor entirely, by passing ozonized air containing 0.3 per cent. of ozone for fourteen minutes through it.

The glaze lining for the pipes, etc., is necessary on account of the corrosive action of ozone. It cannot be stored on account of its instability. A thorough contact of water and ozone is of prime importance. It attacks the easily oxidizable matter first, hence its action upon bacteria is modified by the presence of less resistant substances. Bacteria associated with little masses of solid matter in water are difficult to destroy.

Taking the cost of ozonization at $6.00 per million gallons, and $10.00 per million for filtration, the total cost per million gallons for the double treatment will be $16.00.

The author considers that there lies open for this process a promising field in water purification,—e. g., with iron in the case of turbid water, in the removal of color, odor, etc. Careful experimental study is necessary in order to determine the conditions upon which ozonization can be most effectively and economically applied.

R. S. L.

## DISCUSSION.

MR. MARCEAU remarked that when Mr. Lea gave his valuable paper, on the purification of water, last year, he made some remarks on a new process of sterilization invented by Messrs. Henri and Albert Bergé, of the University of Brussels and stated at the time that he had written the above gentlemen for fuller details of the system. He, some time ago, received from them four pamphlets illustrating their method and containing various analyses of different waters treated by it. He had made an abstract of these pamphlets which he wished to communicate to the Society.

The chemical reagent used was chlorine bi-oxide $CLO^2$, which was most easily prepared by decomposing chlorate of potassium through the agency of sulphuric acid at an average temperature of 15° Centigrade, the formula of the reaction being

$$3\ CLO^3\ K + 2\ SO^4\ H^2 = CLO^4\ K + 2\ SO^4\ HK + 2\ CLO^2 + H\ O^2$$

It was claimed that this chlorine bi-oxide, which had a decidedly destroying effect on all bacterial forms of life, was absolutely innocuous so far as man, animals and plants were concerned. A very small quantity of it was required to perfectly sterilize even waters heavily charged with organic matter and bacteria; the cost of sterilizing a cubic meter of water, Mr. Bergé assured, being as low as ¼ of a centime or even 1-10 of a centime. A cubic meter being equal to 2,200 gallons, an expenditure of 25 francs, or $5.00 per diem would purify Montreal's water supply.

As he remarked last year, this was almost too good to be true, and there must be a hitch some place in the working of the process, otherwise it would now be almost universally used as it had been given to the public two years ago.

It was also stated that waters containing a large proportion of suspended matter should be filtrated before being treated by the peroxide, as otherwise the energy of the oxidizing agent was partly spent on destroying these substances and did not produce its full effect on the germs which were to be destroyed.

The above were the claims of the inventors of the system. Let us see how they were confirmed by analysis.

The City of Ostend, Belgium, had had a series of experiments made to ascertain whether the Bergé process was applicable on a large scale. The water used was that actually supplied to the city. For the purposes of the experiments a polarite filter was placed near the pump house and samples of the natural, filtered

and sterilized water were analyzed chemically by Messrs. Petermann and Bergé, and bacteriologically by Messrs. Van Ermaugem, de Molinari and Bergé, Mr. Bergé in both cases operating separately.

The results of three tests were given in the pamphlets which he submitted. Only an abstract of the conclusions of the city engineer and the general inspector of Ostend would be given.

The method was said to be efficient if operations were made carefully, and regularly. The peroxide, acting during 15 seconds, destroyed almost one half of the organic matter in the water, as well as all micro-organisms. The quantity of reagent required was so small that the quantity of chlorine in the sterilized water was not perceptibly increased.

It was desirable to send the water to be treated through settling basins and to filtrate it, if it were heavily charged with organic or mineral matter. But in the case of a water of proper chemical composition, and which is only dangerous on account of the possible presence of pathogenic germs, such water may be treated by the peroxide process without filtering.

Other experiments were made by Dr. Desguin, president of the Royal Academy of Medicine of Brussels and president of the Medical Commission of the Province of Antwerp, with a view to ascertaining the hygienic qualities of water sterilized by the peroxide process. The following were his conclusions:

I. Water sterilized by the peroxide process has all the organoleptic qualities required as regards taste, colour, smell, and chemical reaction.

II. It is free from all pathogenic microbes, the only bacterias found in it are absolutely innocuous ones, such as "bacillus subtilis."

III. Where used for drinking purposes it always proved innocuous.

IV. It does not interfere with any organic function, germination, fermentation or digestion; assimilation goes on with its use as well as, and sometimes better, than with the use of ordinary waters of good quality.

V. Hygienically, it is perfect.

These results, if *bona fide* as they appeared to be, were close to marvellous, and, as the question of pure water was one of vital importance, he would suggest that experiments and analyses, such as were made at Ostend might be made on Montreal water by some of our members, or by the chemists and bacteriologists of McGill University. To all appearances this could be done at a very trifling cost which the Society could defray if necessary, and, if the results obtained were even half as satisfactory as claimed, our City Fathers might be induced to make a test of the process on a larger scale.

Thursday, 31st January.

MR. G. H. DUGGAN, Vice-President, in the Chair.

On motion, it was resolved to postpone the reading of correspondence in discussion of Mr. Baillairge's paper on "Masonry Dams."

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Thursday, 14th February.

MR. W. MCLEA WALBANK, M. CAN. SOC. C. E., in the Chair.

A series of Stereopticon Views of Engineering Structures on the Canadian Pacific Railway, shewn by Mr. Heckman, occupied the evening.

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Thursday, 28th February.

MR. G. H. DUGGAN, Vice-President, in the Chair.

It was moved by Mr. W. J. Sproule, seconded by Mr. H. Irwin, and unanimously resolved, that "In view of the fact that the 'Review of Reviews' has abused the liberty of free speech, which we all prize so highly, by circulating accounts of dishonourable conduct on the part of British officers and men towards the women of South Africa, which have not been established as true; and by contributions of its editor, Mr. W. T. Stead, to foreign newspapers, ridiculing our present King. even before he had performed any official acts to criticize; and by a general course of action tending to lessen the prestige of our Empire and to incur the contempt of other nations; it is the opinion of this meeting that the said 'Review of Reviews' should be struck from the subscription list of this Society forthwith, and that any numbers already contracted for should be destroyed as soon as received; and that this resolution be sent to the Council and to the Library Committee."

It was then moved by Mr. J. G. Kerry, seconded by Mr. W. McLea Walbank, and unanimously carried:—"That whereas Dr. H. T. Barnes has accepted an invitation from the Paper Committee to lecture before the Society on 'Ice Formation in Canadian Waters and the Physical Laws Governing its Formation,' and has expressed his desire to deliver the lecture in the Physics Building of McGill College, as suitable apparatus for illustrative experiment is available

there, that it be resolved to hold the ordinary meeting of March 14th in the Physics Building for the purpose of hearing the said lecture, provided that no objection to so doing be raised by the College authorities."

Abstract of papers upon "The Character and Qualities of Portland Cement," and upon "Experiments on the Artificial Acceleration of the Setting of Portland Cement," was read and briefly discussed.

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## ABSTRACT.

Abstract of two papers, read before the Liverpool Engineering Society, by Messrs. Prosser A. H. Shaw and F. E. Priest, M.M.Inst.C.E., on "The Characteristics and Qualities of Portland Cement," and on "Experiments on the Artificial Acceleration of the Setting of Portland Cement, respectively.

Both of the papers are evidently written by gentlemen who have had experience in the use of this material.

In the first paper the importance of fineness of grinding is particularly emphasized, and its effect upon the cementitious value is discussed at some length; incidentally it is pointed out that fine grinding, as also the process of air slacking, both have the effect of reducing weight, and, in speaking of the value of weight as a criterion of quality, the author shows that insistence upon a certain standard weight may sometimes lead to the acceptance of a hot (or imperfectly air-slacked) cement, or of one of inferior fineness.

Variations in the weight of samples of the same cement, weighed at different ages, are given, as follows:—

| | | | | | |
|---|---|---|---|---|---|
| At age of | one day, | weight | 117 | lbs. per | bushel. |
| " " " | one month | " | 113 | " " | " |
| " " " | two " | " | 108 | " " | " |
| " " " | twelve " | " | 103 | " " | " |

And, as examples of variation in weight due to grinding to different degrees of fineness, the following are quoted:—

| | | | | | | | | | lbs. per bus. |
|---|---|---|---|---|---|---|---|---|---|
| A cement leaving | 10 p. c. | residue | in an | 80 | mesh | sieve | weighed | | 110 |
| " " | 20 p. c. | " | " | " | " | " | " | | 116 |
| " " | 25 p. c. | " | " | " | " | " | " | | 121 |
| " " | 35 p. c. | " | " | " | " | " | " | | 123 |

The author considers that the chief value of chemical analysis, as capable of application by the average user of cement, lies in the

easy detection, by its means, of the presence of sulphur, or magnesia, in dangerous quantity; beyond this, the tests which he regards as of the greatest practical importance are those for (1) Fineness, (2) Soundness, (3) Tensile Strength.

The economy is referred to of securing a finely ground cement rather than one containing a sensible proportion of a practically inert substance, or of particles, the complete hydration of which, in the author's opinion, may be so long delayed as to endanger the integrity of the mass when this hydration finally takes place; at the same time the wisdom is questioned of pushing the demand for fineness to an extreme; firstly, because the manufacturers can meet the demand by using a shorter clinker, and, secondly, because of the practical difficulties in the way of testing the exact fulfillment of such demands.

Not more than 10 per cent. residue, by weight, upon a 5,800 mesh sieve is suggested as a sufficiently exacting test of fineness for practical purposes.

The author's experience has given him the utmost confidence in the reliability of the "Prussian" or "Pat Test," in conjunction with "Faija's Steam Test," as applied to Soundness.

In discussing the value of tests of tensile strength, the quick development of strength which is exhibited by an over-limed cement is noticed, and an instance is quoted in which strengths of 330 lbs. in water and 740 lbs. in air were attained by a certain cement, after 28 days; the briquettes, however, afterwards suffered complete disintegration through subsequent hydration of free lime. The indication of real value, it is pointed out, is that of a fair induration after seven days; the exhibition, say, of a tensile strength of 350 to 400 lbs. per square inch—with an increase of strength of at least 30 per cent. in 28 days.

Attention is called, in the paper, to the effect upon the apparent strength, of the rate at which the scale is loaded in the testing of briquettes. A slow loading from zero upwards generally leads to a lower "breaking weight" than if the test were begun with an initial loading, equivalent, say, to 100 lbs. It is the author's practice to begin with a load of 100 lbs. applied in 15 seconds—examples of these variations are given.

The difficulty is referred to of carrying out "Sand Tests" in accordance with any standard, as sands of quite different character may each of them equally well fulfill almost any washing or sifting tests that can be applied to them. In the discussion upon the paper, however, the importance of the retention of the sand test was a good deal emphasized, notwithstanding its difficulties, particularly as it might often serve to direct the presence of adulterants. The effect, upon the tensile strength, of the addition to the cement of

diluents, such as pure sand, or Kentish Rag, might be hardly perceptible until the further addition of sand had increased the total quantity of inert matter to a certain proportion, but after this proportion has been exceeded, the further addition of sand leads to a much more rapid reduction of strength than is shown by the same additions to an unadulterated cement.

The addition of any adulterants such as sand, slag, ragstone, etc., the author condemns most emphatically; he quotes instances showing the danger of the presence of slag in a cement which is to be used in damp situations, and it is also shown how the addition of such adulterants must of necessity lessen the cementitious value of the material which fills the interstitial spaces in a rightly proportioned aggregate.

The presence of ragstone can be detected by its effervescence with hydrochloric acid, and by the dark green colour of fracture in a briquette, when broken after immersion in water for seven days. A test for the presence of ragstone, formulated by Mr. Yardley, analytical chemist, is given as follows: "Place on a clean silver coin a thin layer of suspected cement, drop a small quantity of dilute $H_2SO_4$ on it $[H_2SO_4 + 7H_2O]$ then rinse with water. If the cement is genuine the treatment with acid will slightly affect the colour of the silver, but if slag is in strong proportion a dark brown stain will be produced on the silver."

In the discussion on this paper difference of opinion was expressed as to the relative order in importance of the three principal tests discussed by the author, viz., Fineness, Soundness and Tensile Strength, and also with respect to the best practical standard of fineness; a smaller residue upon a coarser sieve than a 5,800 mesh received some advocacy, say 2½ per cent. residue upon a 2,500 mesh —the gauge of the wire also to be duly specified.

It was generally agreed that tensile tests, to be truly satisfactory, ought to cover a long period, and ought to reveal a gradual and progressive induration.

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Mr. Priest's paper was a discussion of the observed effects, upon the time of setting of a cement. produced by the solution in the gauging water of such substances as soda, bicarbonate of soda, sugar, salt, and glycerine in varying quantities.

Soda was found to produce a marked acceleration in the setting, when added to the neat cement in the proportion of 1.44 per cent.

The experiments were carried out with two different samples of cement, the chemical analyses of which are not, however, given; in the cases of both, the effects were very similar. The time of setting was reduced, in the one case, from 9.5 minutes to half a minute, and

in the other case, from 8 minutes to one minute; complete setting taking place in both cases in one hour and forty-eight minutes.

As the quantity of soda was increased above 1.44 per cent., the setting in the case of both cements was retarded, though it was still more rapid than when pure water was used in gauging; all of the other above-mentioned ingredients had a decided effect in retarding the process of setting.

Experiments upon the effects of such additions upon the tensile strength had, at the date of the paper, only extended over a period of 50 days; they showed a perceptible diminution in strength in the case of the soda briquettes, but a striking parallelism between the curves of induration (plotted to time ordinates) of the pure cement briquettes and the soda briquettes respectively.

The great value, under some circumstances, of a reliable means of securing a rapid process of setting was commented upon in the discussion of the paper.

The fact indicated by the experiments seems to be that the addition of a small percentage of soda to the gauging water will have a marked effect in the acceleration of the process of the setting of a cement; and, as many cements naturally contain a small quantity of soda, the quantity to be added to produce the best results will prabably vary from 1½ per cent. to 2½ per cent.

W. R. B.

Thursday, 14th March.

MR. G. H. DUGGAN, Vice-President, in the Chair.

**Paper No. 157.**

## ICE FORMATION IN CANADIAN WATERS AND THE PHYSICAL LAWS GOVERNING ITS FORMATION.

BY DR. H. T. BARNES.

The formation and growth, the disintegration and decay of river ice, forms one of the most interesting themes of study which Nature has to offer, for in that we are dealing with a change of state from a liquid, where the particles are in vibration, rotation aud progressive motion, to a solid, where the particles are confined and restricted to definite centres of vibration; from a liquid which is capable of filling any shaped vessel, we pass to a solid which resists deformation. The process of change takes place, as is well known, at a constant temperature. All crystalline substances have a definite melting point and the liquids which form crystalline solids have a definite solidification point. Water is no exception, and we find that water, on being cooled, experiences a fall of temperature regularly, depending on the rate of abstraction of heat until a definite point is reached. At this point further abstraction of heat causes a change of state,—the formation of ice. The converse process, the formation of a liquid, is accompanied not by an abstraction, but by an absorption of heat. To study this question we must consider the processes by which heat can be abstracted or supplied to the material. The physicist recognizes three ways in which heat is disseminated,—conduction, convection and radiation. These three processes are all employed in causing the loss of heat from a body which is above the temperature of its surroundings. This does not mean that radiation, conduction and convection are the agencies of three different kinds of heat, but that, in the ordinary process of cooling, these serve as the method of transfer. By conduction, of course, we mean the transferring of heat from particle to particle of a body by their direct communication, and from body to body by contact. Thus, if we heat the end of an iron rod, the heat is carried along the rod from particle to particle, and is said to flow along the rod. If the end of the rod is placed in water, the heat passes to the water, and the water increases in temperature. By convection we mean the transferring

of heat by the material movement of the matter itself. As an example, when a liquid is being heated the density decreases, and, in consequence, the lighter part of the liquid rises, carrying with it the heat, to be replaced by the cooler liquid which descends. We can simply illustrate the process of conduction and convection, and of change of state from liquid to ice, by a simple experiment which we shall now perform, and, at the same time, introduce the use of one of Callendar's platinum thermometers, which has been used, as you are probably aware, in certain determinations of the temperature of the river water, and to which we shall more fully refer later on. As you probably know, temperatures by means of the platinum thermometer are measured by observing the variation in resistance of a piece of platinum wire. The arrangement is such that the change in resistance is made to give directly on a suitable scale of units, the number of degrees through which the thermometer is heated or cooled. We shall endeavour to show by this experiment with the aid of one of these thermometers, the gradual lowering of the temperature of some water until the freezing point is reached, and also that the change from water to ice occurs at a constant temperature. As long as heat is abstracted from the water, instead of the temperature falling, ice is formed, the amount being directly proportional to the amount of heat taken away.

EXPERIMENT.

Let us turn our attention now to the case of a river which is flowing, and the case of quiet water. Let us start about midsummer when the temperature is about 80° F., and let us imagine that the weather is becoming cooler as the season advances, and, therefore, the temperature of the air is falling. We will find that if we make accurate determinations of the rate of fall from day to day, this fall will take place in a regularly shaped curve. Thus, as the temperature of the water falls more and more towards the freezing point as the months approach the winter months, we find that it will lag slightly behind the temperature of the air, until finally the air temperature will drop below the freezing point of water. On further abstraction of heat, the water will form ice, and from that time on, the temperature of the water will never vary as much as a hundredth of a degree from the freezing point. The temperature of the water will finally rise above the freezing point during the spring months, or as soon as all the ice has vanished. Now, in the case of a quiet river or a lake, the processes which go on are those of conduction and convection. The surface of the river will be bathed by air which is at a lower temperature than the water itself. The consequence of this will be that the water will be cooled, and this upper cooler water will fall and take the place

of the water below, which will rise, being less dense. This will go on until the temperature of the river reaches 39°, at which point there will be a marked change. At 39° the density of the water is a maximum, and any further cooling of the water will result on the colder water remaining at the top, the warmer staying below. The surface of the water becomes colder than the interior, and, arriving at the freezing point, the result is the formation of ice over the surface, and this ice will then protect the water against loss of heat. The ice, however, grows in thickness more and more, and, as it increases in thickness, will become more of a protection to the water. Some idea can be formed of the rate at which this ice would grow in thickness if it be considered that on the underside the ice was continually bathed by water at 32°, and that the air above the ice was at a temperature of 0°, and if this could remain so for a period of three days, we would then have, if the ice was one foot thick, one inch of ice formed on the underside. This is the result which would be arrived at from calculations of the thermal conductivity, and agrees fairly well with experience. This would mean that, if the weather continued below zero, the ice would increase in thickness in about a month, one foot. The effectiveness of ice in preventing the loss of heat is one of the provisions of nature. Now, the action is quite different if we consider open water. Consider the case of a running river such as the Lachine Rapids, or the water above the rapids. There, the water is churned and mixed to its very bottom. The consequence is that the whole mass of the water falls to the freezing point at the same time, and we have, not the formation of surface ice, as for quiet water, but the formation of fine crystals of ice in the water itself. Heat is extracted from the water rapidly where the air temperature is considerably below the freezing point, which must result in the formation of crystals throughout the mass of the river. This agrees well with what is known to be true in the case of running water. These ice spicules will be formed, but their size will depend upon the rapidity with which the water is flowing. Thus, if the water is being very rapidly agitated we will have very minute crystals. If the water is flowing just too rapidly to freeze we will have plates and discs of ice. Hence, if we find certain ice below the rapids we can tell about how fast the water has been flowing when it was formed. There are certain phenomena in connection with this river ice which are well known, and which can be briefly considered. That is to say, the phenomenon of agglomeration, or the phenomenon which we define as the sticking of the ice together into lumps. It is known that if the water is very cold these ice crystals will be found in a sticky state; in a state in which they will adhere to each other and to bodies which are immersed in the water. There is bound to be a difference of opinion in regard to the action

of this ice in agglomerating. There have been a good many measurements made of the temperature of the water during this change, and it has been inadvertently shown that water becomes very much undercooled. Two or three years ago Mr. Kennedy, of the Harbor Commissioners, interviewed Professor Callendar with a view to undertaking some determination, by means of his thermometers, of the temperature of the water, in order to see if it were possible for the water to become as much undercooled as had been apparently measured by other observers. Professor Callendar asked the author to undertake this work, which he was very glad to do, and a thermometer was designed, which works on the same principle as the one which has been shown you. It is a thermometer which was designed to measure very minute differences of temperature. It was thought, of course, that the water could not be cooled much below the freezing point, because it is known from physical reasons that this is impossible. It can be done in the laboratory by quietly cooling it, and thus lower its temperature below the freezing point; but the slightest agitation will cause the mass to solidify, and there it will remain at the freezing point as long as any water exists. As a result of a careful series of measurements for two winters, one from the guard pier and the other from Lachine Rapids,, the author was led to the conclusion that the rate at which the heat was being abstracted from the mixture of ice and water, or was being supplied to the mixture of ice and water, determines its temperature, and if the mixture of ice and water was losing heat rapidly the temperature was slightly below the freezing point a few thousandths of a degree. It was during the time the water was in an undercooled state that the ice was in this sticky condition, the order of the undercooling being about .006 of a degree. Now this very minute reading would be entirely over-looked by an ordinary thermometer. But by these thermometers, which were capable of measuring temperatures to .0001 of a degree, it was possible to obtain very accurate determinations.

During this state of agglomerations the whole mass of the river is thrown into this slightly undercooled state. Correspondingly, when the air temperature went above the freezing point, the temperature of the water went slightly above the freezing point. This must necessarily follow from theoretical considerations, as the author has shown elsewhere.

There is another agency at work to cause loss of heat. Many scientific men consider that radiation does not play a part in river ice formation. There have been some who have held for a long time that radiation did perform an important part, and many who held that radiation did not. Some experiments will now be shown which will tend to indicate that radiation must play an important part. By radiation we mean the direct rays of heat

which are given out by bodies which are at a higher temperature than their surroundings. These rays are in every way similar to light rays. They are capable of reflection, refraction and polarization, just as light rays, and are propagated through intervening space without necessarily warming that space. Air, for instance, is almost completely diathermanous, as Tyndall showed. Water vapour, on the contrary, is not. It absorbs these heat rays, and, in consequence, if the ray of heat is passed through water vapour, it is absorbed and the temperature of water vapour rises. Now, consider what effect this must have on river ice formation. We have heat radiated from the earth into space. This heat passes out with the velocity of light, hence at night we know that the surface of the earth becomes rapidly cooled by the escape of heat rays into space. In a river, such as the St. Lawrence at the Lachine Rapids, we have the radiation going on from the shore at all points; but as water is partially diathermanous, it is capable of transmitting 11% of the heat rays. We must, therefore, have from the bottom of the river itself a certain amount of heat which is radiated. 89 p.c. will be caught in the water but the rest will pass directly through the water into space. The consequence will be that, with the mass of the river at the freezing point, and this heat being continually radiated from the bottom, the bottom will become slightly cooled below the temperature of the surrounding water. The latent heat will be abstracted from the water, and ice be formed on the bottom. This is a phenomenon which is very familiar to all. The presence of ground ice is one that is a distinct feature of the St. Lawrence River in winter, from the Lachine Rapids six miles up as far as Lake St. Louis, and, in fact, in any part of the St. Lawrence where there is no surface ice. The ice grows from the bottom in arborescent forms. It grows upwards in long fingers, and shows that the ice is buoyed up even as it is formed. The density of the ice itself tends to keep it upwards. We may get a rough and general idea of the quantity of ice which would be formed on the bottom of the river, allowing for the water absorbing 89% of the radiation. We can make a calculation of the amount of ice which would be formed in a given time. The thickness of the water will hardly matter as regards radiation as long as the water is clear. If the water is turbid, however, then the thickness of the layer of water above the ground will matter. If the water is turbid, the 11% of the heat rays are finally stopped, and are reflected back again to the bottom. This can be beautifully shown by a simple experiment, but not capable of being shown on a large scale. If we have a vessel of water in which we have fine particles floating, and we concentrate heat rays on one of these particles, 11% of the heat rays pass through the water unabsorbed, that is, the water does not rise in temperature near the particle; but, as soon as the rays impinge on

a particle, they increase the temperature of that particle, and the water around it becomes heated. This was shown by Tyndall. We can form an idea of the thickness to which this anchor ice will form, if we consider that the mean temperature of the earth is about 47°F. and allowing for the amount of heat which will be absorbed by the water, we will have formed, in about six hours of radiation alone, a layer of ice over the bottom 1 inch thick. This seems to be a very small amount compared with the rapidity with which anchor ice forms, and yet when we consider that this layer would be solid ice, it does not seem so small. The ground ice, which is not solid, but grows in long needles, could be compacted in a very small mass. The radiation itself will go on from the surface of the ice as well; thus the formation of the layer of ice on the bottom of the river does not check the radiation of heat, but the ice continues to increase in thickness. Another and very important indication is the effect of the sun's rays. It is well known that on a clear day as soon as sun rises, the solar rays penetrate through the water and bring up this ground ice in masses. As a proof of the influence of the sun's rays, the author made some experiments on a bright day with the bulb of his thermometer at different depths. These measurements give some idea of the depth of penetration. At 5 feet the temperature indicated by the thermometer was .0186° above freezing. At 2 feet it was .0474° and at 8 inches .0741°. By bending the thermometer stem around so as to carry it under some ice the temperature was .0292°. We see that the protective action of the ice was very great because the ice was covered by snow, and fine particles of snow prevent the radiation of heat through them. In the current which was 10 ft. deep, the temperature was .0280° and shewed that this current must have been running under the ice. Shortly after that, with the thermometer about 5 feet in the water, the sun became clouded over and the temperature then fell to 0112°. The converse effect of radiation from the bottom was measured, that is, similar determinations were undertaken in a clear atmosphere at night. These determinations show that the thermometer bulb became cold by the actual radiation from the bulb.

Towards the early morning after the night of radiation, the water was in a slightly undercooled state and the ice was in a state of agglomeration. The question has arisen whether this ice would be capable of sticking to the bottom of the river not allowing for radiation. Many have held that we do not need to introduce this idea of radiation in river ice formation. But it seems that it must be included in a scientific study of ice formation. Why are these fine crystals of ice which are formed by the admixture of water carried to the bottom and why do they adhere to the bottom, not

admitting the possibility of radiation. The question is one which we would have to consider more in detail. It must be concluded, from other observations, that there must be a first layer of ice formed on the bottom by radiation; after that these needle crystals adhere to this first layer of ice. This layer would form very rapidly after the river had reached the zero point. Then the further increase in the thickness would be from the sticking of the frazil crystals to the foundation laid by the action of radiation and the increase in thickness by radiation from the ground ice itself. The author is now introducing the two terms, "frazil" and "ground ice" as if there were two distinct forms. It is important to have the distinction drawn between frazil and anchor ice. We find that confusion exists and that people refer to ground ice when they mean frazil ice, and vice-versa. Let the distinctive terminology be accepted which was introduced in the report of the Montreal Flood Commissioners and which seems to have been over-looked: "Frazil as distinguished "from anchor ice, is formed over the whole unfrozen surface above "and below Lachine Rapids, between Prescott and tide water and " wherever there is sufficient current or wind agitation to prevent "the formation of bordage ice...."

Many have given out the idea that regelation plays an important part in the formation of ground ice; that these fine crystals, coming in contact with the frozen bottom, stick there by regelation. By this we understand the freezing together of two particles of ice by pressure. At the point of contact of the two, the freezing point is lowered. We know that the application of great pressure will lower the freezing point of water. 150 atmosphered lowers the freezing point one degree. If we press together two particles of ice at the point of contact the ice is melted and the temperature of co-existence of the ice and water is lowered. Immediately on removing the pressure, the water which is at the point of contact, is below its normal freezing point; hence this water must become frozen and in consequence the two particles of ice must stick together. How can this affect the formation of ground ice, seeing that regelation requires continued pressure at one point and how can it act in sticking these ice crystals to the bottom? As a very beautiful experiment illustrating this phenomena of regelation and one which Prof. Cox has been in the habit of showing in his lectures for many years, there has been arranged a block of ice around which has been placed a wire and attached a heavy weight. At the point of contact the pressure is increased and the ice melts and water is formed. This water solidifies again. At the point of contact of the wire, the ice is melting but solidifying again immediately above.

As an example of the growth of frazil crystals which occurs in the Lachine Rapids, we have here several charts of the Montreal Flood

Commission which were made in 1887 and which show that for miles below the point at which the ice is formed we have these minute spicules carried under the ice and passing into quiet water, rising and adhering to the surface. In many places the ice is solid to the very bottom. We have often 20′ of surface ice by the accumulation of these minute crystals carried by the running water under the ice which forms at the foot of the Rapids.

Now, there are many plans which may be formed in regard to alleviating the effects of the agglomeration or sticking together these crystals, but that must be left to engineers and those who are more competent to deal with that side of the question. The author has examined into this question purely from the point of view of a physicist, and, in order to arrive at any results which are going to help in this direction, the question must be studied from the physicists's standpoint, for the accumulation of facts by casual observation will not yield very much. We must look at it from a purely physical standpoint and as a purely physical change, and when we consider the temperature changes which bring about the production of ice, then we will get more nearly to the correct result; but if we cannot entirely obviate and remove the formation of ice in our river here, we may at least find some means to temper its effects.

The steadiness of the temperature of the water is a matter of very great importance indeed. The fact that the water itself does not vary throughout the winter to as much as a hundredth of a degree, is an important consideration. The disastrous effects which result from the formation of frazil to machinery and other engineering work, must be due to the fact that this ice is agglomerating or sticking together in lumps and is liable to adhere to machinery. Now, seeing that the difference of temperature is so small between the disastrous effect of frazil and the condition under which frazil is harmless, why is it not possible to devise some method of entirely obviating the effect of the agglomeration, by suitably warming the water or keeping the machinery (as has been suggested) at a slightly elevated temperature. One way, an important way, of obviating the effects of frazil is to cause surface ice to form in order to cover the river entirely. If a river could be entirely covered then there would be no more disastrous effects from anchor ice and frazil crystals, for the surface ice would then act as a protection. One of the first things that must be done is to cause the cover of surface ice to form. Hence the building of a dam above works is most essential, and, when carried sufficiently far, will prove to be quite effective. Where the danger will lie is that, if the water is left open still above the dam, or place of quiet water, then the frazil crystals may pass in under the ice with the water and gradually

block up the passage. The question of warming the water is a much more serious one. Although it is only necessary to keep it a hundredth of a degree above the freezing point, yet when we consider the mass of water and the fact that water has such a high thermal capacity, it may be realized what an enormous quantity of energy would be required to warm it that small amount. Hence it seems that the only way that the effects of frazil can be obviated is to cause a surface layer of ice to form. As is well known, anchor ice is not formed under bridges. Now, this is a direct indication of radiation. The bridge spanning the river acts as a check to the radiation and causes the radiation of heat back again to the bottom and prevents the formation of ice at that point. Another indication that radiation is a very important feature is, of course, the fact that we find that anchor ice occurs more readily on dark rocks than on light ones. The author has found in the morning that at the place where his observations were made, there was a very much thicker formation on the dark than on the lighter rocks, and it is well known that dark objects radiate the heat more than light ones.

## DISCUSSION.

MR. SPROULE thought that the first thing to be done would be to move a vote of thanks to Dr. Barnes for his extremely interesting lecture. The question, bearing on the subject of winter navigation of the St. Lawrence, was a live one at the present time. The one point of very great interest was that the water did not fall several degrees below the freezing point, as had been supposed by engineers for a great number of years, and as had been recorded by observers at different times and places. It was easy to find records taken for months, from one year to another, showing water temperatures two and three degrees below the freezing point. Dr. Barnes had pointed out many things that were of great interest to engineers, and the facts he had indicated were the ones engineers had to work on in the question of ice and winter navigation. The matter of radiation seemed to be of very great importance. He impressed upon the members present the debt they owed to Dr. Barnes. The information given had not been obtained in a comfortable room, and the observations were not a pleasant job. He had the honour of assisting Dr. Barnes in getting the apparatus together, and the information given represented a good deal of labour and exposure. He had much pleasure in moving a vote of thanks to Dr. Barnes for his very able and interesting lecture.

PROF. DURLEY, in seconding the vote of thanks moved by Mr. Sproule, asked Dr. Barnes, with reference to the figures which he had given as to the temperature of the water while radiation was going on, which of these figures corresponded most nearly to the true temperature of the water.

DR. BARNES, in reply to Prof. Durley's question, said that the lower figure represented more nearly the temperature of the water.

MR. KENNEDY asked how the earth could radiate cold into the water when the bottom was receiving heat from the earth.

DR. BARNES said that in this regard we had to consider the thermal conductivity of the soil. If we took the determinations, which had been carried out by Prof. McLeod and Prof. Callendar, we would find that the thermal conductivity was very small, in fact, it need not be taken into consideration. The radiation from the surface in absolute measure, allowing for the absorption of the rays by the water, would be sufficiently above that of the slow conduction of heat from below to allow for the formation of one inch of solid ice at the end of about six hours of radiation. He said that, of course, these numbers were only general. It would depend altogether on the surface of the bed of the river. The case was for a dark surface.

A QUESTION was raised as to whether there was any difference in the formation of ice in different qualities of water, such as water that would be called hard and water approaching distilled water.

DR. BARNES, in reply, said that soluble substances in water lowered the freezing point. There was no doubt that if water contained a great deal of soluble matter its freezing point was lowered. The quality of ice would not differ, because it was known that from quite impure solutions we had pure ice separating out. Ice separated and expelled the impurities, so that we would have the ice existing in the same clear state as if it were formed from pure water.

PROF. MCLEOD said that in regard to the question, which he supposed related to the fact that some rivers formed frazil and anchor ice more readily than others, he would suggest that that quality would depend upon the clearness of the water rather than upon its chemical quality.

MR. PARIZEAU asked Dr. Barnes if there were any way of distinguishing between crystals of anchor ice and frazil. When the anchor ice detached itself from the bottom and rose towards the surface it became mixed with the frazil, and the two could not then be distinguished.

DR. BARNES, in reply, said that there was a great deal of confusion existing as to the two terms "anchor" and "frazil" ice, and he would like to see the distinction made, based on their methods of formation. Anchor ice was formed on the bottom, growing upwards, while frazil was formed in the river itself. When the two became mixed it was difficult to tell them apart. Frazil crystals seemed to form around particles of suspended material in the water, which formed nuclei for the crystals, and consequently the crystals might be examined for these fine particles, which would identify them as frazil.

A question was raised as to the limit of depth of formation of anchor ice.

DR. BARNES said that, in this connection, there were two factors to be considered. First, as to depth. It was known that anchor ice did not form below a certain depth. The limit was placed at about 30′ to 40′. In the Niagara River anchor ice was unknown. The amount of heat which would be absorbed would depend on the clearness of the water, so we would not have 11% passing through, but perhaps only 5%. And then again, we must remember that in a deep river there was not such thorough stirring of the water as we had in a shallow river, hence the temperature of the water on the bottom of a deep river might not be necessarily at O. We had then the two factors to consider when we went below a certain depth. First, the fact that the water might not be so thoroughly stirred. and, second, that we never had a river with perfectly pure water.

DR. BARNES remarked that, with regard to the use of the terms

"frazil" and "anchor ice," he felt very strongly, having been somewhat interested in the question of the formation of ice from the physicist's standpoint. He had been continually confused by the fact that observers were using the two terms indiscriminately, stating one when they meant the other. It seemed to him that it would be wiser at this stage, more especially as the subject was being more and more discussed, for this Society, which had a distinct and recognized standing in the country, to adopt two distinguishing names for the two types of ice. But this brought up the question, "Do these two forms of ice really exist?" and the point had been raised that it was impossible to distinguish them. We had ice formed on the bed of the river, anchor ice, and ice formed throughout the mass of the river, which was termed frazil. Anchor ice, when detached from the bed of the river, floated down the river and became mixed with the frazil. But he did not think that the distinction was required for ice in the state of disintegration, but it was required where we were regarding ice in its actual formation, and this was the most important feature. Ice which formed on the bed of the river might be formed by radiation and might also be formed by the crystals which were grown directly on the bed of the river, these crystals being attached to stones, etc. The frazil crystals were entirely the consequence of keeping the surface from forming. Surface ice was nature's covering to prevent the loss of energy from the water itself. It was perfectly natural for the ice to form on the surface and prevent further increase in the quantity of ice. Where the river was kept open we had an abstraction of heat which kept going on in the water, and ice, which would otherwise have formed on the surface, formed as crystals of varying degrees of fineness throughout the mass of the river. The word "frazil" might then be applied to the ice which was formed through the mass in the river, and the word "anchor" applied to ice which had grown directly on the bottom of the river. He would like to see the Society adopt these two terms, and believed it would lead to less confusion in the future. We had to recognize the primary difference in the formation of the two kinds of ice, and, as the basis for this, he would refer again to the suggestion which was given in the report of the Montreal Flood Commission.

Mr. Kennedy remarked that, in reference to the adoption of the two terms, he did not see just how we could manage it. We had not taken in hand the matter of making terminology yet, and he did not know how we could go about it. The fact of drawing attention to the usefulness of the two terms was a great deal in itself. Anyone wishing to look into the matter would look up our Transactions and there see the matter referred to.

Mr. Lesage asked how it would be possible to distinguish slush ice from frazil. For instance, what term would be applied to the

ice that formed at the head of the Rapids and lodged itself in the still water basin below?

DR. BARNES said that it would be called frazil. Anchor ice was formed entirely on the bottom. The ice which was found under the surface ice and blocked the channels could not be called anchor ice.

MR. KENNEDY observed that the question would also arise as to what we were going to call ice found under the surface ice. In some places it extended from the surface right down to the bottom, as at Varennes.

MR. KERRY said that the question of frazil and anchor ice was one of a small change in temperature. The actual amount of ice which could be formed by radiation in the course of a day could not be very great, because, according to the laws of radiation, the rapidity of heat loss depended entirely on the difference of temperatures. Dr. Barnes had estimated for us that something like 3″ could be formed over the surface of a river bed in a single night. If there were a considerable amount of anchor ice formed in this way, the melting effect of the radiation of the sun's rays in the succeeding day would be very much greater, because the difference in temperature between the sun and the river bottom was very much greater than the difference between the temperature of the latter and that of the interstellar space. The larger part of what was known as anchor ice seemed to be simply frazil which had attached itself to a layer of ice which was formed on the bottom in the first place. Most of this anchor ice was found in water which was running at a considerable speed, and it must have very considerable cohesive power to attach itself to stationary objects as it did. The frazil, which was found underneath the surface ice, was not frozen together, and had very little cohesive power. The actual difference between frazil and anchor was that anchor ice was undercooled and had the power of anchoring itself, which power it lost with rise of temperature. So far as practical engineering problems were concerned he thought they arose mainly from frazil and not from anchor ice.

MR. MARCEAU wished to know what was the practical necessity of distinguishing between the two forms of ice. He thought that the nature of anchor was almost exactly the same as frazil. Water held in suspension a number of minute particles which got colder by radiation and acted as a nucleus around which these small needles formed. The same process went on at the bottom, and he thought that the nature of the two was alike. As soon as anchor ice was detached from the bottom it became frazil.

MR. IRWIN said that the frazil that he had seen looked solid, and seemed to be of a whitish colour. He did not see why we could not distinguish anchor ice as ice that is actually stuck to the bottom.

DR. BARNES remarked the anchor ice was more solid than the frazil crystals. In drawing a distinction, of course, they were both ice, and both had the same chemical composition; but the use of

the two words would make it easier for future observers to record observations and make it easier to describe the phenomena.

MR. KERRY thought that we had already accepted Dr. Barnes' suggestion, and every speaker had taken it that frazil was detached material, and anchor ice was anchored.

MR. SPROULE remarked that, as he understood the matter, Dr. Barnes' object was to get at the way in which the ice was formed. There was no doubt that what was anchor ice in one place was frazil in another, assuming that what was formed on the bottom was called anchor ice. No doubt a large proportion of the ice that was contained in Laprairie Basin and elsewhere had been formed on the bottom at Caughnawaga and Lachine, and was true anchor ice that had been formed there. Anchor ice was formed by radiation on the bottom of the river, and was supposed to form by far the greatest quantity of the loose ice that was termed frazil. He did not understand that anchor ice was anchored to the bottom of a sheet of ice. He would suggest, as a distinction, that anchor ice was only ice that observers had reason to believe had been formed on the bottom of the river.

DR. BARNES observed that a great deal of the ice that was formed in the mass of the river was formed when anchor ice was not rising at all. It was only when anchor ice commenced to disintegrate that it became mixed with the mass of the river.

MR. SPROULE remarked that there would be one or two words necessary to describe the spicules which were formed in the water and the thin surface layer which was formed on the surface of the water, so thin that it was difficult to recognize it. There would be four kinds of ice: anchor, frazil, the fine spicules and the thin sheet.

MR. KERRY said that he would like to take issue with Mr. Sproule as to the relative quantities of frazil and anchor ice. Speaking from figures, which Dr. Barnes gave us, it was perfectly clear that the actual amount of anchor ice which was formed on the bottom was very small indeed, compared with the whole amount of ice which was formed by the undercooling of the water. If there was a certain amount of anchor ice formed in the night by radiation it was melted in the day by radiation also. On the other hand, if the temperature was considerably below 0, the formation of frazil, from the undercooling at the surface, was going on day and night. In certain parts of the day there would be no anchor ice being formed in situ.

MR. ROBINSON observed that if we undertook to determine the character of the ice from its origin we would only perpetuate the confusion of terms for the reason that no two observers would agree to the origin. He, therefore, asked if it would not be simpler to agree that anchor ice was that which was anchored, and the ice which was not anchored was frazil.

MR. SPROULE said that the relative quantities of anchor and frazil was a point that he was not aware was settled. At one time he was of opinion that by far the greater part of the ice was formed in the mass of the water. He thought that the assumption that the ice, formed as anchor ice, was formed in the night was a decided mistake, because he was perfectly sure that the great masses of ice which he had seen rising from the bottom were not melted during the day, and he should judge that a vast quantity of ice during the day rose up from the bottom and did not disappear, but was carried under the surface ice. He thought most people would agree that ice was formed in the mass of the water only in extremely cold weather, and only when anchor ice would be formed during the night.

MR. KERRY remarked that Mr. Sproule had misunderstood his estimate of the actual amount of ice formed as anchor ice. The estimate referred only to that formed at the river bottom by radiation at night, and the amount of this was estimated to be considerably less than the amount that would be melted by the sun's radiation on the subsequent day. The great masses of anchor ice, to which he referred, were, in his opinion, formed by the adhesion of frazil to a small amount of ice previously formed in situ by radiation.

## ABSTRACT.

### WINTER TEMPERATURES OF WATER.

BY DR. H. T. BARNES.

Trans. Royal Soc. of Canada, 1896 and 1897.

In these two papers the author describes the methods and results of a series of measurements of the temperature of the water in the St. Lawrence River. The 1896 observations were taken from the Guard Pier, in Montreal harbour, and the 1897 observations from the embankment of the Lachine Rapids Hydraulic Company.

The apparatus used was an electrical resistance thermometer, the temperature being determined by measuring the difference of resistance to the passage of an electric current offered by two similar wire coils, one of which was immersed in the river, and the other in a carefully prepared and frequently renewed mixture of snow and ice. The temperature of such a mixture is known to be exactly 32° Fahr., and the thermometer determined the temperature of the river above or below that of the mixture. The author states that in a laboratory, measurements can be made to 1-10,000 of a degree cent. But owing to the difficulties arising from the exposed situation and character of the observation shanties, the actual measurements made are not regarded as accurate to anything less than 1-1000 of 1° C.

For engineering purposes the results of these observations may be stated briefly as deciding that throughout the winter the water has an exact temperature of 32° Fahr., no variation as great as 1-100°C. above or below that value being observed.

The 1896 observations extended over the period between Feb. 15, and March 16, and were made through holes cut in the surface ice, and the 1897 observations were made Feb. 3, and the middle of March, mostly in the open water of the Lachine Rapids. The author describes at some length the apparatus used and the method of making the observations, and gives instances of the minute rises of all the water temperatures that followed marked changes in the air temperatures and in the meteorological conditions. So dependent, indeed, were these minute changes upon the air temperature and meteorological conditions that the author states that it was possible to predict them with considerable accuracy.

The author's conclusions are that the formation of frazil follows a slight under cooling of the river water, and is accompanied by a slight rise of temperature, due to the liberation of the latent heat upon crystallization. This undercooling is partially due to the direct abstraction of heat from the water surface, by conduction into the air and partially to radiation. The formation of anchor ice as distinguished from frazil is ascribed to radiation from the river bottom into the atmosphere, especially on clear cold nights, but the author admits that the masses of anchor ice may grow rapidly by frazil particles carried by the river currents becoming attached to the anchor ice formation. The rising of the masses of anchor ice is said to be due to the heating of the river bottom by the radiant heat in the sun's rays.

Conclusion from experiments would seem to be that river is a natural storage of a certain amount of heat gathered in the summer and gradually lost throughout the winter, by conduction and radiation. If the losses due to these actions were severe enough, the river would become solid. The most important agent in checking them is the formation of surface ice. It seems desirable that the Society should advise that a series of experiments be conducted to determine, if possible, the rate of loss due to different temperatures when the water is open and when covered by ice and snow of various thicknesses. The Lachine Canal might be a suitable position for such experiments. It is certainly necessary to measure the strength of the adverse forces that militate against the strength of our engineering works.

**ABSTRACT.**

## ICE FLOODS AND WINTER NAVIGATION OF THE LOWER ST. LAWRENCE.

By T. C. Keefer.

Trans. Royal Society, Canada, 1898.

In this address, the author gives a short history of the Montreal floods of various dates, and describes, at some length, the manner in which the ice forms on the river below Montreal, and the facts about the winter condition of the river. The information gathered in 1886 and 1887, by the Montreal Flood Commission, of which the author was chairman, is particularly referred to, and reprints of several of the plans prepared for that Commission accompany the paper. The surveys for the Commission demonstrated that between Montreal and the foot of Lachine Rapids, there was at least as much frazil ice as water, and for twenty miles below Montreal the frazil blocked 30 per cent. of the waterway. This frazil is said to be formed as anchor ice in the open water. A change of temperature loosens its hold in the bottom and it is carried underneath the solid ice lower down and adheres to it, growing rapidly in volume as the spiculae of frazil attach themselves to it.

The ice bridge commences to form at Lake St. Peter and advances up the river to Montreal as rapidly as descending bordage and drift ice furnish material for it. The river is really packed with broken ice rather than frozen over, and underneath this solid surface gathers the anchor ice and frazil, blocking up all the channels and causing the floods. In Lake St. Louis, the frazil has been found as deep as 85 feet, and causing a reversion of the current of the Ottawa between Cascades Point and Vaudreuil.

The two recommendations of the Flood Commission were that an attempt should be made to close the open water above the Lachine Rapids by means of a lumber boom, which, by checking the floating ice, would aid in the establishment of an ice bridge and reducing the frazil forming area; and that the ice bridge below Montreal should be prevented from forming by ice breaking tugs, so that both bordage and frazil might pass straight in to the Gulf waters, the open channel thus secured being the surest safeguard against the recurrence of floods. On the recommendation of the Commission, the Flood Dyke was built in Montreal, and as there has never been a flood in the streets of Montreal since that construction was finished, their other recommendations have been forgotten, but the author

is still of the opinion that it is perfectly practicable to maintain an open channel throughout the winter from Montreal to the sea, and that it ought to be attempted not only with the view of flood protection, but also because of the great value to the St. Lawrence River ports of a lengthened season of navigation. The paper concludes with some illustrations of old time Montreal ice shoves and some remarks upon American and Russian ice breaking steamers.

Thursday, 28th March.

MR. E. MARCEAU, Vice-President, in the Chair.

Illustrated address by Mr. John Kennedy on "The Construction and Operation of Ice-Breaking Steamers in Their Application to Navigation of the St. Lawrence in the Winter."

## ABSTRACT.

## STEAMERS FOR WINTER NAVIGATION AND ICE BREAKING.

BY ROBERT RUNEBERG, A.M.I.C.E.

(Proceedings Inst. C. E., Vol. CXL., page 109.)

Before 1877, Finland was closed to navigation for five months in the year. Now, communication is kept up, practically speaking, the year round, with regular trade steamers between Hangö, England, Germany, Sweden, and other countries. During the last few years considerable modifications and improvements have been effected in the design and construction of ice-breaking vessels. Built according to the European type, they break the ice by the sloping bow rising until its weight overcomes the resistance of the ice, which then breaks, the bow descends, and the steamer progresses again to elevate the fore-end on the edge of the ice. When the ice is too thick to allow steady progress the boat retires a few hundred feet and charges against the ice at speed. With the gradual increase in the power of modern ice-breakers, evenly laid and uniform ice no longer presents any serious hindrance to winter navigation. In the Baltic the usual thickness, where there is no pack-ice, seldom exceeds 18 to 24 inches, even in severe winters. An ice-breaker of 2,500 I.H.P., with the propeller aft, but otherwise of modern design, should be able to work its way through this ice, if not covered with snow, without stopping, and if covered with snow she will progress fairly by charging.

The author quotes the "Aegir" as a modern type of ice-breaking vessel. She was built in 1896 by the Nylands works at Christiania, and, though small, has done excellent service. Her leading dimensions are:—

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|---|---|
| Length over all.. .. .. .. .. .. .. .. .. .. .. .. | 182 ft. |
| Length between perpendiculars.. .. .. .. .. .. | 172 ft. |
| Breadth.. .. .. .. .. .. .. .. .. .. .. .. .. .. | 28 ft. |
| Depth, moulded .. .. .. .. .. .. .. .. .. .. .. | 14 ft. 6 in. |
| Dead weight at 16 ft. draught.. .. .. .. .. .. | 500 tons. |

She has a flush upper deck and double bottom, and especial strength in stem, stern-post and rudder.

It had been observed that a steamer going astern sometimes made better progress than when going ahead, and accordingly the "St. Ignas" was built in 1888, from designs by Mr. Frank E. Kirby, by the Detroit Dry Dock Company for the Michigan Central Railway Company, for the purpose of carrying cars across the Straits of Mackinac; her leading characteristic being a front propeller and her principal dimensions:—

| | |
|---|---|
| Length over all.. .. .. .. .. .. .. .. .. .. .. .. | 235 ft. |
| Breadth.. .. .. .. .. .. .. .. .. .. .. .. .. .. .. | 51 ft. 6 in. |
| Depth.. .. .. .. .. .. .. .. .. .. .. .. .. .. .. .. | 25 ft. |
| Draught.. .. .. .. .. .. .. .. .. .. .. .. .. .. .. | 15 ft. 8 in. |

The engines are compound, the aft engine developing 2,000 I.H.P. at 90 revolutions per minute, the fore engines 1,000 I.H.P. at 100 revolutions; the steam pressure is 120 pounds per square inch. The diameter of the aft propeller is 12 ft., with 16½ ft. pitch, and the fore propeller 10 ft. with 15 inch pitch, both being of cast steel. The hull is built of wood sheathed with ¼ inch steel plate to above the load water line. The fore propeller was made extra strong, and it should be noted that no propeller blade of the "St. Ignas" has been broken. Once the fore propeller was entirely lost, but the reasons have not been made known.

The method by which this new craft worked in pack-ice was somewhat different from the European plan. When an ice-breaker of the old school was stopped by an ice belt she backed astern a few ship lengths and make a fresh charge, which always brought her a little way further in the ice. The "St. Ignas" rests with the bow against the ice; throwing the fore engines back the propeller sends a stream of water among the pack-ice, loosening the grip between the separate blocks, and as soon as the engines are again reversed the lumps of ice are carried aft by the stream from the fore propeller. The aft engines, during this operation, are constantly working forwards, and, having a greater power than the fore engines, keep the boat against the ice.

This ferry-boat having proved a success, the "St. Marie" was built in 1892-93, somewhat more powerful and longer, in order to take seventeen or eighteen cars, whereas the "St. Ignas" carried only ten. Further, the ratio of the power of the fore and aft engines was changed, the fore engines being given the greater part of the increase, or, in all, 800 H. P., at 100 revolutions per minute, while the aft engines developed 2,200 H.P. at 90 revolutions. The compound system was again adopted. The principal dimensions of the "St. Marie" were as follows:—

| | |
|---|---|
| Length | 304 ft. |
| Breadth | 50 ft. 6 in. |
| Depth | 24 ft. |
| Draught | 17 ft. |

In 1892 the Craig Shipbuilding Company of Toledo built two smaller ferry boats for the Frankfort-Keraunee Co., named "Ann Arbor I." and "Ann Arbor II," the principal dimensions of "Ann Arbor I." being:—

| | |
|---|---|
| Length over all | 267 ft. |
| Breadth | 52 ft. |
| Depth | 18 ft. 6 in. |
| Draught, loaded | 12 ft. |
| Draught, light | 9 ft. 10 in. |

The fore-engines, compound, 550 H.P., at 95 revolutions per minute. Aft-engines, twin screw compound, 1,160, total H. P. at 93 revolutions. Aft-propeller, diameter 9 feet, and fore-propeller, 8 feet 3 inches.

The "Ann Arbor II." has a slightly larger hull, but the same engine power and diameter of propellers. The good results obtained with the American Railway Ferry boats were soon appreciated in Europe. One of the most difficult and expensive parts of the Trans-Siberian Railway is along the Great Baikal Lake, and it was therefore decided to follow the example of the Americans and build a powerful steam ferry boat to carry the trains across the lake, leaving the construction of the line round the shore for a future operation. The Russian government subsequently ordered a steam ferry boat of the American type to be built by Messrs. Armstrong, Whitworth & Co., of Newcastle, in the year 1895, with dimensions:—

| | |
|---|---|
| Length over all | 290 ft. |
| Breadth on main deck | 57 ft. |
| Breadth on water line | 53 ft. 6 in. |
| Depth | 28 ft. 6 in. |
| Draught in fresh water, laden aft | 20 ft. |
| Draught in fresh water, laden forward | 17 ft. |

She is to carry 25 cars, weighing 500 tons, with fuel, stores, spare gear, passengers and luggage, 250 tons. The indicated H.P. is to be not less than 3,750.

Further particulars of this ice-breaker and several others, with plates, are given in the paper. Of the larger and more modern vessels of this character may be mentioned the "Ermak." ("Engineering News," New York, 1899, No. 2, page 31; No. 17, page 265. "Engineering," London, 1899, 24th February and 31st March.) She is 305 feet long, 71 feet broad and 42 feet 6 inches in depth.

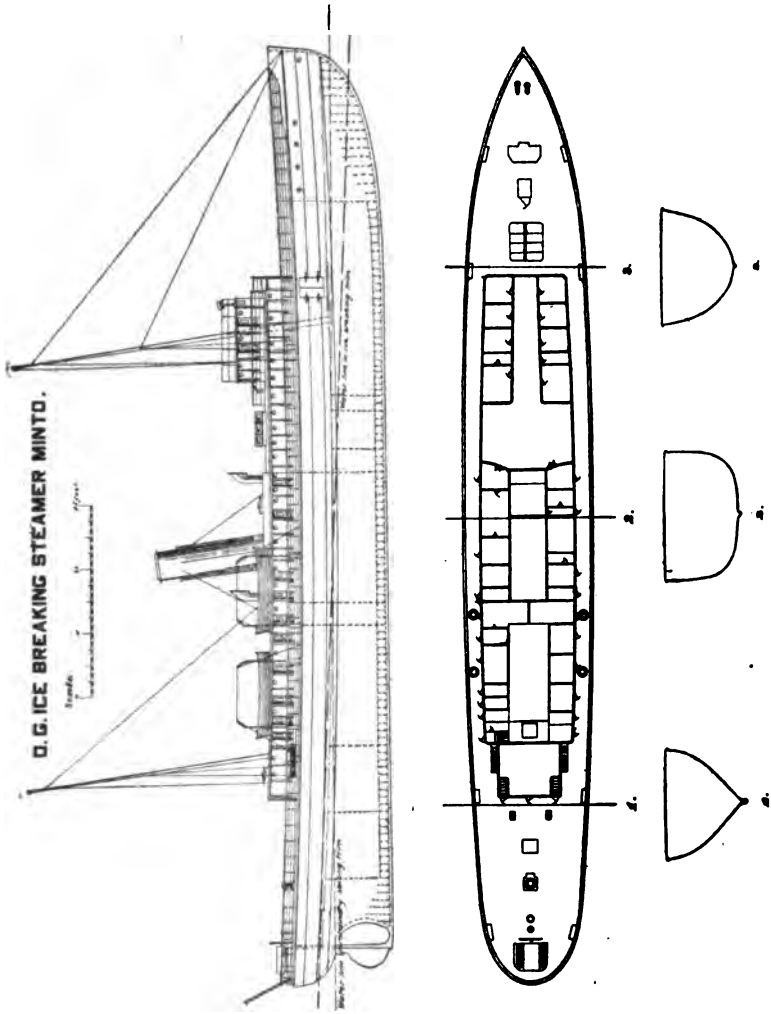
The ice-belt is 20 feet broad, and between 1¼ and 1 1-16 inches thick, the maximum thickness being amidships. The main frames are spaced at 24 inches, but intermediate frames are fitted in the way of the ice-belt, and Webb frames are furnished at intervals between the bulk-heads. There are four sets of triple expansion main engines, driving three screw propellers aft, and one forward. Each shaft is arranged to be disconnected from the main engines, and geared to an auxiliary compound engine, the intention being to work only the auxiliary engines when full speed is not required, and thus save fuel. The advantages of this arrangement over the more simple way of working only one or two main engines is not very apparent, however, and complication in the way of machinery is not desirable in a vessel of this description. The propeller blades are of nickel steel, and all shafts are from 35 per cent. to 60 per cent. stronger than required by Lloyds. The total power is, according to the contract, 10,000 H.P. The boilers are double ended, placed athwart-ships, their diameter being 15 feet, and their length 20 ft. 6 in. The heating surface is 27,600 square feet, forced draught being used. The stern propellers are 14 feet in diameter, the central propeller having a pitch of 14 ft., and the wing propellers a pitch of 14 feet 6 in. The forward propeller is 13 feet in diameter, with a pitch of 13 ft. 6 in. The vessel can carry 3,900 tons of coal. The power of the fore-propeller is only 25 per cent. of the total power. According to the American experience it is desirable to have the power nearly equally divided, or say 45 per cent. on the fore-propeller.

Reference is made to a former paper by the same author on the same subject (Minutes of Proceedings Inst. C. E., Vol. XCVII., p. 277), in which is given (a) an exhaustive treatment of the theoretical considerations governing the resistance of ice, and the action of ice-breaking steamers; (b) the points of practical importance in their construction, and (c) full particulars as to dimensions, etc., of numerous vessels of this type, with notes of their performance under varying conditions of ice and weather.

Both papers are accompanied by plates showing lines, sections, etc., of the vessels described.

It is thought that these papers will, at this juncture, have a special interest for the Can. Soc. C.E., in view of the steadily increasing proportions of the trade of the Commercial Metropolis of the Dominion, the great drawback to its progress due to the closing of its navigation for five months in the year, and the tremendous importance of the attempt said to be in contemplation to raise the ice-siege by means similar to those employed with such gratifying success in the waters of Northern Europe.

C. E. W. D.


O. G. ICE BREAKING STEAMER MINTO.

## DISCUSSION.

MR. MARCEAU said that he was sure that the navigation of canals in winter in this part of Canada was quite impossible. The ice formed early in November, and by the end of the month, it might be about 4″ or 5″ thick, but sometimes did not form until the middle of December. It would be quite possible to break this ice, and he had seen boats come through the Lachine Canal with 4″ and 5″ of ice, but it was hard on the boat to get through it, and harder to handle the lock gates, for the ice accumulated behind them. As the ice became thicker, it would take a great deal more time to lock the ice than to lock the boats. If water was kept open in the canals, frazil was sure to form and block the sluices. He thought that in a current of about a mile an hour, frazil would form if the water were open. He was sure frazil would form in greater or less quantity.

MR. ROGERS thought that on the Trent Canal, navigation could be kept up through the winter. On an isolated lock it might be possible, but with a number of locks he did not believe it could be effected in a practical manner. In the spring, when the ice was 3′ or 4′ thick, the only way to get rid of it was to allow it to melt.

In the fall, some boats came down about the beginning of December. It might be possible to keep the canal open until the middle of December, but only a few boats ever passed down after the end of November. Last year there were three or four boats that came down on the 4th of December. In the spring, the month of May was the time of opening. The canal was emptied for necessary repairs, and the ice must be allowed to melt. This year the ice was 4′ and 6′ thick in many places. The canal was practically clear of ice about the middle of April.

MR. KERRY said that he noticed in regard to the first locks going up from the harbour that the lock at Black's Bridge was nearly always clear of ice through the whole winter, while in the lock immediately above it, the ice must have been 4′ thick. He asked for an explanation of this.

MR. MARCEAU observed that the return pipe from the Montreal Street Railway works emptied into the canal near the lock referred to and discharged hot water, thus preventing the ice forming.

MR. J. L. WELLER asked if it were the ice that stopped navigation in the fall and early winter on the great lakes. He was of the opinion that the ice had very little to do with it, and that if it were the only drawback to navigation, the problem would soon be solved, and we should see vessels navigating the great lakes well into the

new year if not all winter. As a matter of fact, very few vessels stayed out after the first week in December, although it was well known that there would be no ice for another month that an ordinary steel boat would not go through with ease. There was practically no ice in Lakes Erie and Ontario until well on in January. The closing of navigation was due to a well defined feeling of insecurity on the part of the Insurance Companies, vessel owners and masters. Because vessels navigate the ocean with safety in the winter, and enter ice bound ports, it did not follow that they would be able to do so on the lakes; heavy winter storms prevailed there, and our boats were not built for these storms, and in our lakes, large as they were, there was absolutely no sea room to ride out a snow storm of any duration. The season of navigation on our canals could, no doubt, be lengthened if there were any necessity for it, but at present there were only a few vessels going through them during the last two weeks in the fall. By keeping up a moderate current in the canals, the formation of ice was much retarded, and there should be several weeks difference in the date of closing a stagnant canal and one in which the flow of water was judiciously governed, and in which boats were moving continuously.

Mr. R. B. Rogers remarked that as regards the formation of ice on, or the winter navigation of the St. Lawrence, he could not speak from knowledge, but as the success of winter navigation on the Trent Canal must depend upon that of the St. Lawrence, he gave the following notes on the subject. It was intended that the main part of the traffic on the Trent Canal would be carried by tows of barges. The route was through lakes and rivers. The lakes varied from ¾ to six miles in width, from one to twenty miles in length, and from 10 to 20 feet in depth, and included Lake Simcoe, which was a small inland sea. The width of the rivers varied from 399 to 600 feet, and their depths from 8 to 16 feet. The lakes and parts of the quieter portions of the rivers froze over and the ice had a thickness of about twelve to twenty-four inches by the middle of February. The first freezing over took place about the 15th or 20th of November, and this ice was about 4 or 5 inches thick. This stopped navigation, but, as a rule, the ice melted in about a week's time, or became much thinner, and navigation was again open for two or three weeks. The ice, up to the 1st of January, was not more than ten to twelve inches in thickness. If a current was allowed to escape through the valves, no ice would be formed about the lock or lock-gates to prevent the working of the gates, even in the severest weather. He saw no reason why navigation should not be kept open from one end of this canal to the other, from the 1st of April to the 1st January— a period of nine months—at a very small cost. He could see no physical reason why navigation should

not be kept open the whole year around, but whether it would be a commercial success would have to be determined by experience. There was one element in favour of an all year navigation, that was that the wages of vesselmen would be much lower in summer than at present, if engagements for the whole year were made. The first ice on our lakes, where the water was still, with an air temperature of about five to ten degrees below zero in the night, and of about five to ten degrees above zero in the day, formed to a thickness of about 4 inches in 36 hours. Another such night and day would not increase the thickness over two inches, and a third such day probably not over 1¼ inches, the rate of increase in thickness of ice depending on the temperature. In our rivers, where there was a perceptible current, the ice never attained any great thickness, and was never safe to drive over; this was, owing to the warmer water coming from the bottom of the lakes, which was warmer than the surface water below the ice and wore away or prevented increase in thickness of the ice already formed. To keep navigation open till the 1st of January on this canal, no extra or special construction would be required beyond sheeting the bow end of the leading boats with iron, and having some simple device, such as an 8″ "I" Beam attached to the bows to run upon and break the ice a few feet ahead of the vessel. To break the ice about the 1st of March, would require a boat of somewhat special construction, but it need not be an expensive vessel as the ice was pretty well rotted away by this date. To keep navigation open the whole winter would require some vessels specially constructed for the purpose. As the traffic in this canal was intended to be carried on by tows of barges, the leading vessel or tug would require to be specially constructed.

Mr. C. R. Coutlee remarked that ice formed at the rate of about an inch on the first night, increasing in a couple of days to 6 inches, and if cold weather continued to 10 and 12 inches in December. Its general thickness was about two feet from January to March; but it was found as thick as 4 feet, and sometimes in several layers. In March, thick ice was on lakes, covered with soft snow, 2 feet deep. Crevices had formed due to expansion, and along these lines openings began. At places, surface ice was underlaid with frazil and cakes, which had drifted in and some times packed down to the bottom. This spongy dam lessened cross-section and caused currents at other points, which wore the underside of ice field by attrition of blocks carried beneath. Melting began early in March, and ice gradually covered with water, which made a slush of surface snow, the temperature being + OC. Another source of water on the ice was the creeks and general drainage; this flow was at temperatures slightly above freezing point. "Honey-combing" took

place and the ice sank lower in to the water. This was due to the breaking down of the sides of the ice cells and the consequent escape of the imprisoned air which had buoyed up the ice. Once honey-combed and sunk, a wind, a current or a slight rise in temperature soon broke up the field. A feature of departing ice was the detachment of large flows, which floated about at will of wind or currents. In currents less than 2 miles per hour, ice would take during severe weather. Generally a field floated in and jambed across, forming a boom, against which cake ice was arrested. Frequently the cakes turned on edge, owing to the current forcing them underneath. Winter navigation between Montreal and Quebec was a matter of money. If the trade merited the expenditure, a passage should usually be possible, and some ships might come up or a specially equipped line of steamers might develop. As a flight of imagination the probable work of a season might be described:—

Everything would require to be in readiness for the last week of November. Steel clad propellers, with saucer shaped bows to mount the ice, would be required to patrol the channel. An equipment of each boat would likely be ice plows, ice saws, ice channelling chisels and pounders working from steel booms at ends and sides. A liberal complement of labourers, tools, explosives and plenty of coal would be necessary. Along the open strip the edge ice would be wave-lapped and thickened accordingly, so that large floes would constantly be breaking off. Frazil ice would slip in from beneath the fields alongside and roll along the open slip. This loose ice should be constantly herded along and floes swung across the road should be cut up. Edge ice was also liable to cut the sides of ships. A collection of frazil at any point might render further navigation impossible. The Russian Ermac, a powerful mass of steel, armed with screw agitators and able to force herself at ten miles per hour through thick hard ice, was as helpless on a bank of slush as if she were on a sand-bar. The open strip resembled a very wide crevasse and these were constantly closing, the edges crushing together with tremendous force. Thus the work of days might be nullified in an hour, and a valuable vessel and cargo pinched flat. To allow of berthing at docks, large areas would have to be cleared and constant care exercised to prevent the ship freezing against the wharf face. A shove in the harbour might also cause great damage. At Montreal, the canal navigation would begin. The first difficulty would be to keep cake ice from behind the gates to allow of their shutting back into the recesses, and a still greater difficulty would be to keep frazil out of the lock chambers. This would form in the currents necessary to filling the locks and give no warning

of its presence till everything was blocked with "water pulp." The surface ice in the breaches might be broken by raising and lowering the level, but the broken cakes would have to be got rid of. This ice could not be hauled up on the banks, and even with a fair wind its passage through the locks would be a tedious business. During the early winter the conditions were always worse till the beginning of March; after that they improved rapidly. To assist nature in the spring break-up, therefore, seems more logical, and this had been practiced in inland waters very generally. The ferry Str. "Pierpont" had forced some very early crossings at Kingston, but sometimes the whole field—miles in extent—through which she was cutting, had moved *en masse.* and carried steamer and all off in the direction of the willing wind. The Canada Atlantic ferry, plying between Coteau and Clarke's Island, until 1889, was once carried willy-nilly down stream in a detached field. Great difficulty was experienced from frazil about the docks, which had to be removed by explosives and labourers working day and night. The ice bridge at Quebec, had been removed by explosives. Powder charges were placed in coal oil barrels, which were sunk beneath the ice, the air space of the barrels being pointed in the direction of the required blow. Ice fields of large extent could be moved by a tug pushing bow on.

MR. R. BICKERDIKE, JUN., remarked that it was doubtful if Montreal would ever become, commercially, a winter port. The most that would be expected at least for a great many years, was that the season would extend from March 15 to Dec. 15, or a gain of about 2½ months over present period of navigation. With an ice-breaker of moderate power, there was no difficulty in keeping the river open for 9 months in the year. After March 15 (if the river had been kept open all winter, or opened about March 1), the buoys and lights could be placed and it then became a question of gulf ice alone. Many alterations would have to be made in the ship channel before Montreal would become a winter port, and it was more in this connection than with considerations of ice-breaking, that difficulties might be expected. With a river full of floating ice, buoys and light-ships were useless, and in winter, many of the shore marks, on which the pilots were supposed to depend, were so altered in appearance as to be unreliable. At present the channel was being improved by eliminating some curves, widening from 300 to 450 feet (wider at curves), and deepening to 29 feet at the lowest water on record, giving a gain of from 3½ to 4 feet. With the channel improved as above, and with permanent marks and an ice-breaker continuously at work, the channel, from Montreal to Quebec, should be navigable all winter. Between Montreal and Quebec there were four critical points for ice bridge formation, viz.: Sorel Islands, Pointe du Lac, Grondines and Cap Rouge; and if ice jams were pre-

vented at these places, ice of two feet thickness would practically never be encountered. With a jam in Sorel Islands, ice piles in places 10 to 30 feet thick.

MR. ST. GEORGE BOSWELL remarked that within the last few years, the public of Canada had become interested in the question of winter navigation of the St. Lawrence. This was largely due to the fact, that on the Baltic, and on our great lakes, it had been practically demonstrated that the presence of ice offered no insurmountable obstacle to navigation. In regard to the winter navigation of the St. Lawrence below Quebec, the opinion of experienced pilots and navigators, familiar with the condition of the river in winter, was usually in favour of the project—(*Vide*, "Quebec Morning Chronicle" of Jan. 2, Jan. 10, and Jan. 16 instant). They stated that from the Traverse down, there was open water to be found along the north shore, the prevailing currents and winds holding the ice against the south shore; and that the only danger to be apprehended was that from a vessel being caught in a snow storm, while in the Traverse; and they considered that, with mooring blocks erected at various points above and below the Traverse, to which vessels could be moored during a snowstorm, there would be no difficulty in winter navigation, with well built ships. In fact, successful winter trips to the Lower St. Lawrence had already been made on several occasions, and this by vessels not well adapted for the purpose. In one instance a steam schooner, with small propelling power, left Quebec on the 12th of March for the Strait of Belle Isle, and reached her destination without difficulty. In another, a similar schooner left for Murray Bay on the 16th of Dec., and returned on the 24th of March. Again, a steam schooner left on the 22nd of December, and one on the 17th of February, bound for the Gulf, which was reached without any hindrance from ice. Between Quebec and Levis three ferry boats were kept in service during the winter months, and were able to make the crossing in all weathers, this too in an easterly gale of wind, when the floating ice became packed against the Quebec side of the river, making an almost solid mass of ice, one half a mile wide, through which the boats had to force their way. Below Quebec, where the river opened out and was more directly east and west, the ice did not become packed in the same way, and would not offer a serious obstruction to a well built and fairly powerful steamship, should she have to force her way through it; this would, however, seldom happen, as there was generally a fairly open channel. The greatest difficulty with the ice would probably be at the entrance to the Gulf, between St. Paul Island and Cap Ray. Here, owing to wind and current, the ice from the St. Lawrence and Gulf became blocked, and occasionally formed a so-called bridge across the straits. This "bridge" did

not, however, form before the middle of April, and so would affect the present summer navigation, and not the winter navigation under discussion. (See "St. Lawrence Pilot," Vol. 1, 1894). The conclusion, which the available information on the subject would seem to point to was, in his opinion, that winter navigation of the St. Lawrence up to Quebec was entirely feasible, under the following conditions:—

1st. That vessels, somewhat more strongly built than for ordinary purposes, be employed in the service.

2nd. That all floating marks, lights, buoys, etc., be replaced by permanent structures.

3rd. That mooring piers be constructed at suitable points, above and below the Traverse.

4th. That a pilot station be established where vessels could moor and be in security in all weathers.

He had never seen frazil ice in the St. Lawrence River at or below Quebec.

MR. J. M. SHANLY observed that in the Baie des Chaleurs, as far down as Paspebiac, the ice formed solid along the shores and in the bays, but a short distance out the water remained perfectly open until about the middle or end of Jan'y.; after that floating ice began to gather, and sometimes extended as far out as could be seen, but it was always in motion and never seemed to be in very large masses, so that a steamer, if properly equipped, would have very little trouble in getting through it. This ice disappeared entirely about the beginning of March. When the "Gaspesia" made her first trip, a few days before Christmas, the Bay was as clear as at midsummer. On her second trip, in January, she got in all right, and on leaving, found no difficulty until well out in the Gulf, when near the Magdalen Islands, she ran into an ice floe and remained there for six or eight weeks, and apparently a few miles to the northward there was open water all winter. The whole question of winter navigation seemed to him to depend on the Gulf itself; and no one seemed to know how the ice acted there, but there were a number of vessels which went there every winter after seals and seemed to get about without much difficulty.

MR. J. S. ARMSTRONG remarked that the following notes in reference to ice in the Shubenacadie River had been used in connection with a specification for a railway bridge, but might be of interest as a record generally applicable to the rivers near the head of the Bay of Fundy. Here the tides swung back and forth with the regularity of a pendulum, and rose and fell about equally above and below mean sea level, but varied with the influences of sun, moon and wind. The flux and reflux varied in height and increased from

the open ocean towards the head of the bay where the extreme play was said to be from 50 to 60 feet. The highest record known to him was 45 feet at the mouths of the La Planche and Missegash Rivers, close to the boundary between Nova Scotia and New Brunswick. These tides rushed very rapidly through the narrower waters and poured in and out of the rivers whose beds were so much above the elevation of low tides, that they were left nearly bare for a considerable time. At the mouth of Five-Mile River the tide ran at its greatest rate, about three quarters of an hour after it commenced to come in, where the water had an elevation from 10 to 15 feet below extreme high water and a velocity of nine miles an hour. The ice in the Bay of Fundy and the Strait of Canso was not generally like that in the St. Lawrence at Quebec—not nearly so hard and brittle, more like snow ice, except that the bay ice was largely coloured with red mud. During the last two years the ice had not been as heavy as it was often claimed to be in the Strait of Canso. The ice there usually originated in Bay St. George and the Gulf of St. Lawrence, and was brought down with a north-west wind and a tide setting south, and often formed a jam at McMillan's Point, the narrowest place on the strait, leaving the strait south of that practically free of ice. Then again in the spring with south-east winds there was liable to be a flow of what was locally called the southern ice, which was ice that had worked round the eastern end of Cape Breton, or the far heavier ice from the Arctic that came round Newfoundland, and sometimes streamed southward as far as Halifax. The smaller ice when driven by the wind packed very solidly, piling piece upon piece on the lee side of the flow which rendered it very difficult for a vessel of moderate power to work out, if caught between the flow and a lee shore.

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Notes on the Shubenacadie ice:—

About four miles above the mouth of Five Mile River, near Cove Creek or River Side, the river freezes over and remains so until spring. Below that an ice wall forms on either side of the river, and between these walls the ice forms into blocks. These blocks grind against each other until they settle and become frozen down to the flats. They then begin to grow thicker by the deposit of snow (which is wetted by the tide and turned into a porous ice), and by other blocks settling on top and freezing to those below. Mud and sand are carried on to them by the tide. Sometimes during the winter they lift from the flats, but when they are large this does not take place until the tide is nearly high, and the current is not strong. When they lift from the flats, they bring up a lot of mud and sand. So they grow in weight and height, and when floating they do not rise much above the water.

The ice walls form the heaviest ice, but they do not lift except with the very highest tides, and they do not float off and get into the current until they are much wasted and broken up, in the spring. They are then honey-combed and rotten. In the blocks there are sheets, layers or parts that are hard, while other parts are much softer. During the winter the only ice that floats freely in the current is in the channel, where ice of a moderate thickness runs back and forth continually. Sometimes in the fall very large sheets of salt water ice come up the channel—perhaps an acre in extent—but they are not like the blocks that form on the flats. They are supposed to be thin, soft and slushy. The heavy ice gradually wastes away and works out in the spring; only a few of the large lumps work into the channel and are carried out, never coming back again. The largest lumps are, say 20′ x 30″ x 15″. Most of the ice that runs out in the spring is what comes from where the river freezes over, and when this breaks up it is rotten and becomes very much broken before it reaches the mouth of the Five Mile River. In the Shubenacadie, where the Railway line crosses near the mouth of the Five Mile River, the channel is of a well-defined width, extending from the Rocky Bank a quarter of the way across. The channel at this point never varies from this position either in winter or summer.

Thursday, 11th April.

MR. G. H. DUGGAN, Vice-President, in the Chair.

The following donation to the Library was reported:—

"Water Supply Engineering," from Walter M. Scott, S. Can. Soc. C.E.

The discussion on "Winter Navigation Through Ice" occupied the evening.

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Thursday, 25th April.

MR. H. IRWIN, Member of Council, in the Chair.

Messrs. W. McL. Walbank, E. S. M. Lovelace, W. Chase Thomson, J. G. Legrand. W. A. Clement, H. Irwin and R. A. Ross, having been appointed scrutineers of the ballot for the election of members, reported the following elected:—

MEMBERS.

H. D. BUSH.
A. A. DION.
F. H. GROSE.
H. E. T. HAULTAIN.
W. C. JOHNSON.
W. MURDOCH.

ASSOCIATE MEMBERS.

A. G. JOANNIDIS.
A. T. KERR.
G. D. MACDOUGALL.
F. G. MACKIE.
O. W. SMITH.
D. WEATHERBE.

ASSOCIATE.

U. P. BOUCHER.

Transferred from the class of Associate Member to the class of Member:—

C. R. COUTLEE.
E. S. MATTICE.

Transferred from the class of Student to the class of Associate Member:—

J. A. DOUGLAS.
E. P. GOODWIN.
A. R. HOLMES.

STUDENTS.

G. E. BELL. A. C. D. BLANCHARD.
L. T. BRAY. J. A. HEAMAN.
Y. D. KIRTON.

Messrs. E. S. Mattice, S. P. Robbins, G. R. Macleod and E. A. Rhys-Roberts, having been appointed scrutineers of the ballot for the Sunday opening ballot, reported as follows:—

Members in good standing in favour of opening the rooms on Sundays, 94; those opposed, 58.

Members in arrears in favour of opening the rooms on Sundays, 78; those opposed, 14.

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**Paper No. 158.**

## ON THE EFFECT OF TIME AND TEMPERATURE ON THE STRENGTH OF STEEL AND IRON.

BY ERNEST GEO. COKER, B.A. (CANTAB.), D.Sc. (EDIN.).

### INTRODUCTION

The behaviour of iron and steel under simple stresses of various kinds has long been a subject of investigation by the Engineer, and very complete data are available for use in designing structures in metal.

When iron or steel is subjected to tensional stress sufficient to cause a permanent stretch in the material, it is well known that the ductility of the material is reduced, and that ultimately the elastic limit rises.

As a first consequence, however, the elastic limit is first of all lowered, in some cases to zero, but if the bar be left to itself the elastic limit ultimately rises to a higher value than before.

In the last few years the phenomenon has received considerable attention at the hands of Bauschinger*, Ewing† and Muir‡.

The present paper relates to a few experiments which have been made in the Testing Laboratory, McGill University, following the lines indicated by the authorities mentioned above.

### I. MEASUREMENT OF SMALL STRAINS.

The measurement of the change in length of a bar, by such a small quantity as 1/50,000 inches has been rendered comparatively

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* Unwin's Testing of materials of construction.

† Ewing "On the measurement of small strains in the Testing of materials and Structures" Proc. R. S. 1895.

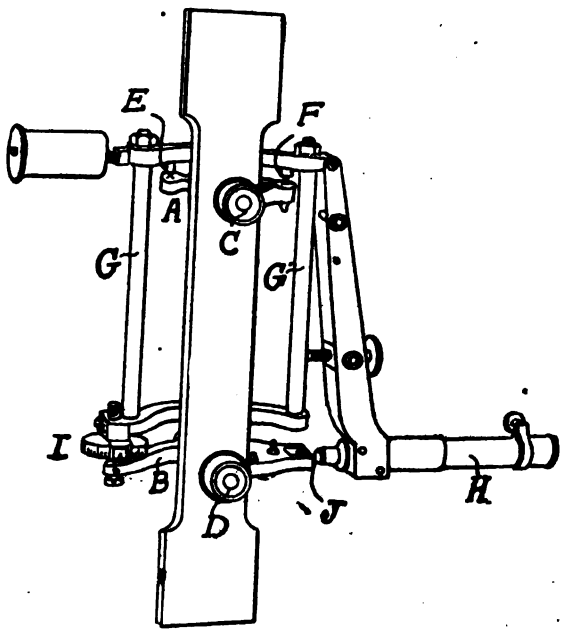
‡ Muir "On the recovery of Iron from overstrain." Phil. Trans. 1899.

easy by aid of extensometers devised by Bauschinger, Unwin and Ewing. The instrument mainly used in these experiments was one constructed after Professor Ewing's design, and has several novel features. It is, moreover, especially adapted for work where several settings of the instrument are necessary.

## II. EWING EXTENSOMETER.

The instrument is shown in Fig 1, mounted upon a test piece, and consists of two clips, A and B, fixed upon the bar at a standard distance apart by set screws C, D. Upon the upper clip, A, are two pins E. F., upon which latter the frame G is mounted. The frame carries a reading microscope H, and a micrometer screw I; this latter gears with a conical hole in the lower clip B, and is kept in engagement by a spring. The reading microscope is sighted upon a thick wire mounted horizontally in a plate, J, and illuminated from behind by a small mirror.

Fig 1



The clips are made of gunmetal, and are formed so as to give slightly when the set screws engage the test piece, and thereby if a slight diminution of section occurs in the bar, while the test is proceeding, the screws do not become slack, and one source of error is thereby avoided.

The clips A and B are each centred upon the bar, and each has one degree of freedom with respect to it, viz., that of rotation about their respective set-screws, and by causing the micrometer screw I to gear with the lower clip, both degrees of freedom are removed and the apparatus becomes a rigid whole.

In setting a flat tension bar in the jaws of a testing machine, it is difficult to avoid giving it a slight twist, but by pivoting the frame G upon the upper clip in a transverse direction, a slight twist will not cause error in the readings.

The micrometer screw and observation wire are set equidistant from the set screws D upon the clip B, so that the wire can be moved up or down by turning the micrometer screw, and this movement is measured by reference to a glass scale in the eye piece of the reading microscope.

In this manner the scale is calibrated by reference to the micrometer screw, which latter has 50 threads to the inch, and the scale is so arranged that a movement of the wire through 1-50 inch corresponds to a reading of 50 scale divisions. In actual working since the micrometer screw engages the clip B, any extension between the set screws will cause the relative movement between the reading microscope and the wire to be doubled, so that the value of one scale division $=$ 1-2 x 1-00 x 1-50 $=$ 1-5,000 inch, and as 1-10 division can be estimated, a reading of 1-50,000 on an 8 inch length can be measured.

The chief advantage of the instrument is that it can be calibrated in position, that the reading is a mean of two sides of the bar, and that it can be quickly and accurately set on the specimen.

In order to set the clip accurately upon the specimen a distance piece is provided, having V shaped recesses to receive corresponding studs on the clips, and clamps to hold the clips in place, until these latter are set on the bar.

The apparatus for applying tensional stress was a single lever Wickstead testing machine, which was capable of exerting a pull of 200,000 pounds. As the stresses used were comparatively small, the weigh lever was mounted to cause the jockey weight to travel the whole length of the weighbeam for a range of 50,000 pounds, and so render the measurement of applied stress as accurate as possible.

## III.—RECOVERY OF STEEL FROM TENSIONAL OVERSTRAIN.

When a bar of iron or steel is subjected to a tensional stress, sufficient to cause a permanent stretch in the material, the physical

character of the bar changes, and it becomes semi-plastic, while its elastic limit generally reduces to zero, followed by recovery, which is comparatively rapid in the case of wrought iron, but less so in the case of steel.

As an example of this, the case of a mild steel bar may be taken. This bar, and others referred to later, were cut from a piece of boiler plate, and afterwards straightened and annealed. The bars were machined to a breadth of 2.07 inches, and the thickness was left untouched, the mean thickness in the present case being 0.377 inches, and the length under test 8 inches.

The Extensometer used was of the Ewing type, one division corresponding to 1-50,000 inches.

A test was first made to give the bar a permanent stretch, with the following results:—

| Loads in Lbs. | Reading of Extensometer. | Difference. |
|---|---|---|
| 0 | 400 | |
| | | —35 |
| 2,000 | 435 | |
| | | —35 |
| 4,000 | 470 | |
| | | —34 |
| 6,000 | 505 | |
| | | —35 |
| 8,000 | 539 | |
| | | —34 |
| 10,000 | 573 | |
| | | —35 |
| 12,000 | 608 | |
| | | —35 |
| 14,000 | 643 | |
| | | —36 |
| 16,000 | 679 | |
| | | —36 |
| 18,000 | 715 | |
| | | —38 |
| 20,000 | 753 | |
| 23,000 | Went off scale. | |
| | Extended length. | |
| 24,000 | 8.10 inches. | |
| 24,500 | 8.12 " | |
| 25,000 | 8.14 " | |
| 26,000 | 8.15 " | |
| 16,500 | 8.155 " | |
| 27 000 | 8.165 " | |
| 0 | 8.15 " | |

The measuring instrument was immediately re-set to the standard distance of 8 inches, and a re-test was made, rising by increments of 2,000 pounds to 24,000 pounds, and then to zero again by decrements of 2,000 pounds. The following results were obtained:—

| Load in pounds. | Reading of Extensometer. | Difference. |
|---|---|---|
| 0 | 400 | |
| | | —35 |
| 2,000 | 435 | |
| | | —34 |
| 4,000 | 469 | |
| | | —37 |
| 6,000 | 506 | |
| | | —37 |
| 8,000 | 543 | |
| | | —37 |
| 10,000 | 580 | |
| | | —42 |
| 12,000 | 622 | |
| | | —42 |
| 14,000 | 662 | |
| | | —42 |
| 16,000 | 704 | |
| | | —40 |
| 18,000 | 744 | |
| | | —54 |
| 20,000 | 798 | |
| | | —48 |
| 22,000 | 846 | |
| | | —51 |
| 24,000 | 897 | |
| | | —35 |
| 22,000 | 862 | |
| | | —35 |
| 20,000 | 827 | |
| | | —37 |
| 18,000 | 790 | |
| | | —42 |
| 16,000 | 748 | |
| | | —39 |
| 14,000 | 709 | |
| | | —40 |
| 12,000 | 669 | |
| | | —40 |
| 10,000 | 629 | |
| | | —41 |
| 8,000 | 588 | |
| | | —42 |
| 6,000 | 546 | |
| | | —42 |
| 4,000 | 504 | |
| | | 44 |
| 2,000 | 460 | |
| | | —45 |
| 0 | 415 | |

A comparison of the readings obtained at once shows how differently the bar behaves after the tensile overstrain. There is now no linear relation between the stress and the strain, but the latter

rises irregularly as the stress increases; the differences are irregular and do not agree except accidentally with the differences obtained as the load is removed.

The bar was now laid on one side, and a gradual recovery of its elastic properties took place, which was shown by the approach to the first readings made. At the end of three hours there was a marked recovery, and the following readings were taken:—

| Load, lbs. | Reading. | Difference. |
|---|---|---|
| 0 | 400 | |
| | | —34 |
| 2,000 | 434 | |
| | | —34 |
| 4,000 | 468 | |
| | | —37 |
| 6,000 | 505 | |
| | | —37 |
| 8,000 | 542 | |
| | | —38 |
| 10,000 | 580 | |
| | | —39 |
| 12,000 | 619 | |
| | | —41 |
| 14,000 | 660 | |
| | | —40 |
| 16,000 | 700 | |
| | | —42 |
| 18,000 | 742 | |
| | | —45 |
| 20,000 | 787 | |
| | | —42 |
| 22,000 | 829 | |
| | | —46 |
| 24,000 | 875 | |
| | | —34 |
| 22,000 | 841 | |
| | | —35 |
| 20,000 | 800 | |
| | | —37 |
| 18,000 | 769 | |
| | | —38 |
| 16,000 | 731 | |
| | | —38 |
| 14,000 | 693 | |
| | | —39 |
| 12,000 | 654 | |
| | | —41 |
| 10,000 | 613 | |
| | | —40 |
| 8,000 | 573 | |
| | | —41 |
| 6,000 | 532 | |
| | | —43 |
| 4,000 | 489 | |
| | | —43 |
| 2,000 | 446 | |
| | | —44 |
| 0 | 402 | |

The tests of the bar were repeated at the end of one day, 3 days, 7 days and 28 days, and these showed a further recovery, whose rate diminished with the time, but at the end of the month the recovery was practically complete. The readings obtained are shown in the annexed table:—

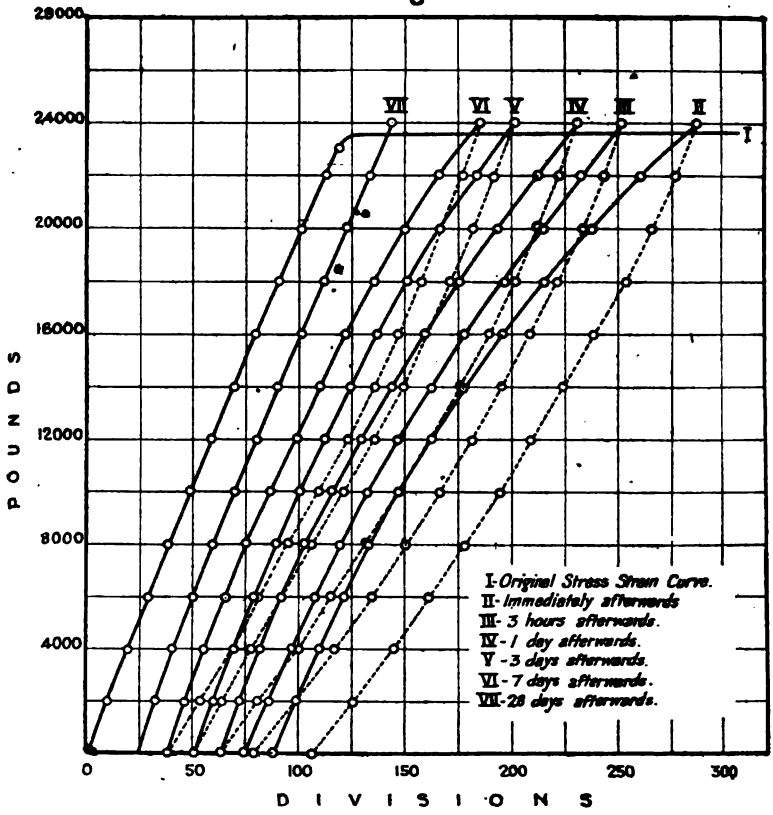
| Load Lbs. | 1 day after. | | 3 days after. | | 7 days after. | | 28 days after. | |
|---|---|---|---|---|---|---|---|---|
| | Reading. | Δ | Reading. | Δ | Reading. | Δ | Reading. | Δ |
| 0 | 400 | | 400 | | 400 | | 400 | |
| 2.000 | 434 | -34 | 434 | -34 | 434 | -34 | 433 | -33 |
| 4.000 | 469 | -35 | 469 | -35 | 468 | -34 | 467 | -34 |
| 6.000 | 504 | -35 | 504 | -35 | 503 | -35 | 500 | -33 |
| 8.000 | 539 | -35 | 539 | -35 | 538 | -35 | 535 | -35 |
| 10.000 | 576 | -37 | 574 | -35 | 574 | -36 | 570 | -35 |
| 12.000 | 614 | -38 | 611 | -37 | 609 | -35 | 605 | -35 |
| 14.000 | 654 | -40 | 648 | -37 | 646 | -37 | 640 | -35 |
| 16.000 | 695 | -41 | 685 | -37 | 684 | -38 | 676 | -36 |
| 18.000 | 736 | -41 | 725 | -40 | 722 | -38 | 712 | -36 |
| 20.000 | 779 | -43 | 765 | -40 | 761 | -39 | 747 | -35 |
| 22.000 | 822 | -43 | 807 | -42 | 802 | -41 | 783 | -36 |
| 24.000 | 867 | -45 | 851 | -44 | 847 | -45 | 818 | -35 |
| 22.000 | 832 | -35 | 817 | -34 | 813 | -34 | 784 | -34 |
| 20.000 | 797 | -35 | 782 | -35 | 779 | -34 | 750 | -34 |
| 18.000 | 762 | -35 | 746 | -36 | 745 | -34 | 715 | -35 |
| 16.000 | 724 | -38 | 710 | -36 | 709 | -36 | 680 | -35 |
| 14.000 | 686 | -38 | 672 | -38 | 672 | -37 | 645 | -35 |
| 12.000 | 648 | -38 | 634 | -38 | 635 | -37 | 611 | -34 |
| 10.000 | 607 | -41 | 596 | -38 | 596 | -39 | 576 | -35 |
| 8.000 | 566 | -41 | 556 | -40 | 558 | -38 | 540 | -36 |
| 6.000 | 525 | -41 | 517 | -39 | 519 | -39 | 505 | -35 |
| 4.000 | 483 | -42 | 478 | -39 | 481 | -38 | 470 | -35 |
| 2.000 | 441 | -42 | 438 | -40 | 443 | -38 | 435 | -35 |
| 0 | 398 | -43 | 397 | -41 | 402 | -41 | 399 | -36 |

In order to exhibit the changes in a manner which shows the recovery in a striking manner, the results were plotted in Fig. 2 after the manner first suggested by Prof. Ewing.* A number proportional to the load was subtracted from each reading, and the new reading obtained was plotted instead; this accentuates the differences, and as each curve is plotted from a different initial point, the recovery is exhibited in a form which can be readily followed by the eye. In the present instance 25 divisions were subtracted for each increment of 2,000 pounds.

Referring to the diagram, curve I. denotes the original stress strain curve, only part of which is shown, as the total exension Table I. is 8,250 divisions at a load of 27,000 pounds. Curve II. and the succeeding curves show the result of the overstrain. The stress-strain curve becomes a loop, and the area enclosed is a mea-

*Muir on the Recovery of Iron from Overstrain. Phil. Trans. 1899.

Fig. 2




sure of the work done on the bar by the loading and unloading, and therefore is a measure of the imperfectness of the elasticity.

The rest caused a gradual recovery of the metal, and showed itself in the gradual diminution of the enclosed area until at the end of 28 days the recovery was very nearly but not quite perfect.

## SHEAR STRAIN.

The recovery of iron and steel from shear strain appears to follow the same general law as for tension. It will be sufficient to quote the case of a turned specimen of wrought iron, having a diameter of .0446 inch, and length under test of 4.00 inches. The specimen was subjected to shear by a twisting couple, and the distortion between the sections was measured by a strain measuring instrument of special design.* The bar was first subjected to torque of gradually increasing amount until the elastic limit was passed. The readings obtained were as follows, the calibration of the instrument being such that 12.8 divisions corresponded to 1 minute of arc.

*Coker "on Instruments for Measuring Small Torsional Strains." Phil. Mag., 1899

| Torque inch lbs. | Reading. | Diff. | Torque inch lbs. | Reading. | Diff. |
|---|---|---|---|---|---|
| 0 | 0 | | | | |
| | | —302 | | | |
| 75 | 302 | | | 1489 | |
| | | —303 | 360 | | —81 |
| 150 | 605 | | | 1568 | |
| | | —303 | 375 | | —79 |
| 225 | 908 | | | 1666 | |
| | | —304 | 390 | | —98 |
| 300 | 1212 | | | 1808 | |
| | | —63 | 495 | | —142 |
| 315 | 1275 | | | 2038 | |
| | | —64 | 420 | | —230 |
| 330 | 1339 | | | 2548 | |
| | | —69 | 435 | | —510 |
| 345 | 1408 | | | Specimen twisted through. | |
| | | —81 | 450 | | |
| 360 | 1489 | | | 4° | |

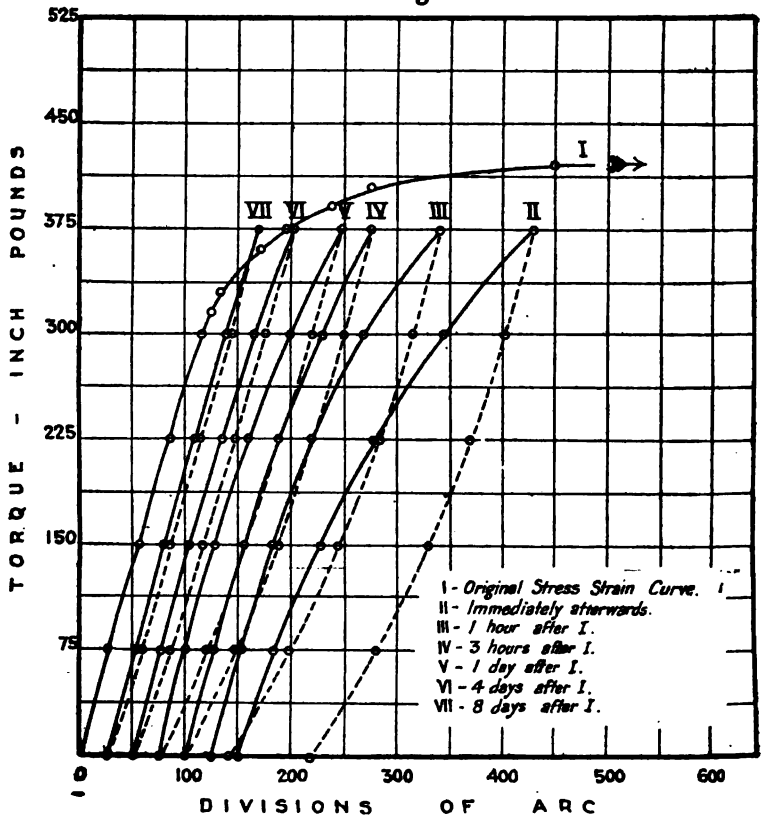
Immediately after the load was released and a new series of readings obtained, the maximum torque applied being 375 inch pounds, and similar observations were recorded at the end of one hour, 3 hours, 1 day, 4 days and 8 days, the last experiment showing practically complete recovery.

The readings were as follows:—

| Torque inch lbs. | Immediately after I | | One hour after I | | Three hours after I | | One day after I | | Four days after I | | Eight days after I | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Reading. | Δ | Reading. | Δ | Reading. | Δ | Reading. | Δ | Reading. | Δ | Reading. | Δ |
| 0 | 0 | | 0 | | 0 | | 0 | | 0 | | 0 | |
| | | —305 | | —304 | | —301 | | —301 | | 301 | | —302 |
| 75 | 305 | | 304 | | 301 | | 301 | | 301 | | 302 | |
| | | —324 | | —306 | | —303 | | —302 | | 303 | | —303 |
| 150 | 629 | | 610 | | 604 | | 603 | | 604 | | 605 | |
| | | —324 | | —312 | | —307 | | —305 | | 304 | | —304 |
| 225 | 953 | | 922 | | 911 | | 908 | | 908 | | 909 | |
| | | —339 | | —319 | | —317 | | —314 | | 307 | | —303 |
| 300 | 1292 | | 1241 | | 1228 | | 1222 | | 1215 | | 1212 | |
| | | —361 | | —349 | | —322 | | —316 | | 310 | | —305 |
| 375 | 1653 | | 1590 | | 1550 | | 1546 | | 1525 | | 1517 | |
| | | —3 3 | | —301 | | —301 | | —302 | | 302 | | —301 |
| 300 | 1350 | | 1289 | | 1249 | | 1244 | | 1223 | | 1216 | |
| | | —305 | | —307 | | —304 | | —303 | | 303 | | —302 |
| 225 | 1045 | | 982 | | 945 | | 941 | | 920 | | 14 | |
| | | —315 | | —312 | | —309 | | —307 | | 306 | | 303 |
| 150 | 730 | | 7 0 | | 636 | | 634 | | 614 | | 611 | |
| | | —325 | | —324 | | —316 | | —314 | | 307 | | 304 |
| 75 | 405 | | 346 | | 320 | | 320 | | 307 | | 307 | |
| | | —338 | | —332 | | 321 | | 319 | | 308 | | 306 |
| 0 | 67 | | 14 | | —1 | | +1 | | —1 | | 1 | |

The readings obtained are given in the above table, and are plotted on Fig. 3, after deducting 275 divisions for each increment or decrement of 75 inch pounds of torque. The rapid recovery of wrought iron is well shown by the looped stress-strain curves II. to VII, and, the rapid diminution of hysteresis in one hour after the overstrain is very marked.

Fig. 3

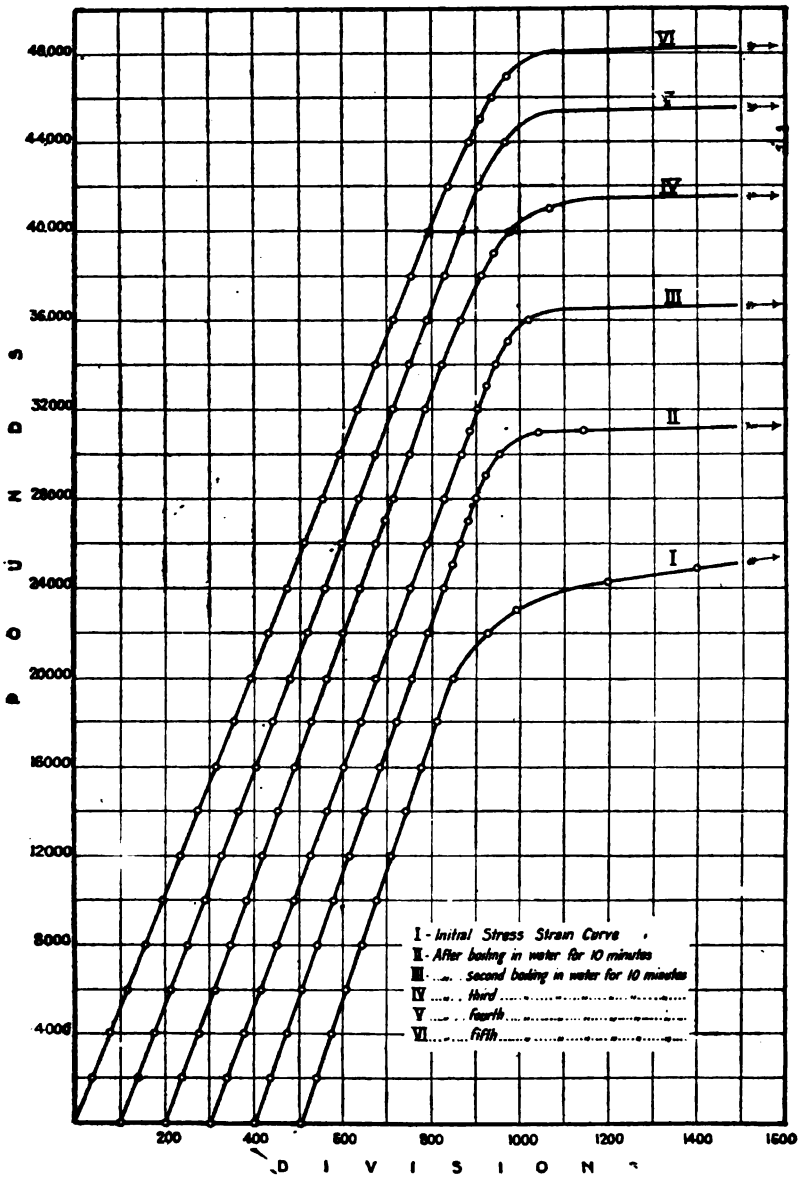



(*Calibration of instrument – 1 minute of arc = 12·85 divisions*)

THE INFLUENCE OF TEMPERATURE UPON THE RECOVERY FROM OVERSTRAIN.

It is well known that if a bar of iron or steel be stressed beyond the elastic limit in tension, it will regain its elastic properties if raised to a dull red heat and allowed to cool slowly in air or in lime, and this process may be repeated again and again without

Fig. 4


48,000
44,000
40,000
36,000
32000
28000
24000
20000
16000
12000
8000
4000
POUNDS
200
400
600
800
1000
1200
1400
1600
DIVISION
VI
V
IV
III
II
I
I - Initial Stress Strain Curve
II - After boiling in water for 10 minutes
III ..... second boiling in water for 10 minutes
IV ..... third .....
V ..... fourth .....
VI ..... fifth .....

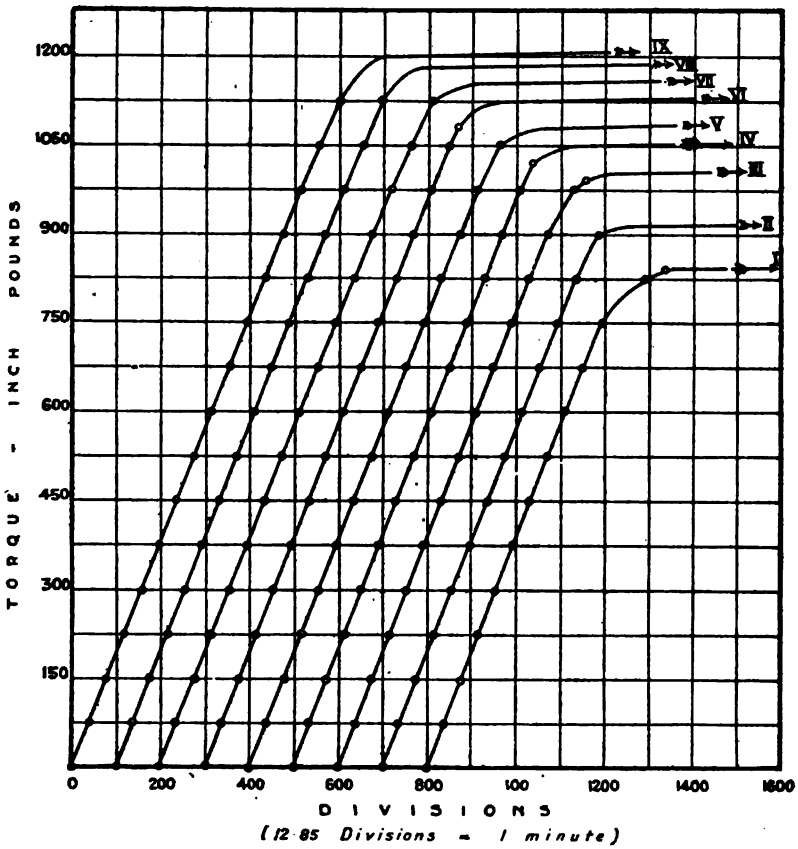
seriously affecting the physical properties of the bar. Recently our knowledge of annealing has been greatly increased by the discovery of Muir* acting on a suggestion of Prof. Ewing, that a comparatively low temperature, such as that of boiling water will restore a strained bar to its elastic condition. As an example of this a test made in the McGill Laboratory shows the effect of boiling water upon an overstrained bar. The specimen was a flat bar of boiler plate, width 2.07 inches, thickness 0.375 inch, and length under test 8.00 inches. This specimen was similar to the one used in the first experiment, and was annealed at the same time. The history of the bar is shown by aid of figure IV., in which Curve I. shows the relation of stress to strain when the bar was subjected to a gradually increasing stress. The first notable deviation from perfect elasticity began when a total stress of 20,000 was applied, and at 25,000 the extension amounted to 8.13 inches, an increase of stress to 27,000 pounds, caused a further extension to 8.15 inches. The load was then removed, and the bar was exposed for 10 minutes to the temperature of boiling water and again re-tested. Curve II. shows the resulting stress-strain curve, with a yield point at about 31,000 pounds, considerably above the first. The boiling was repeated in the same manner, and a new yield point (Curve III.) was obtained at 36,500 pounds, while further tests after annealing in boiling water gave new yield points at 41,500, 45,600 and finally 48,100, when the bar broke, with a total extension of about 15 per cent. This extension is less by about 6 per cent. than would have been obtained if the bar had been stressed to breaking without any intermediate treatment. Even a temperature of 50 degrees C. effects a considerable physical change in a bar of overstrained material, restoring its elasticity and raising the yield point.

## THE EFFECT OF TEMPERATURE ON OVERSTRAIN PRODUCED BY SHEAR.

The annealing effect of boiling water is equally marked upon material overstrained by shear. As an example, a test was made upon a turned steel bar of 0.425 inch diameter, and having a length of 4 inches under test. On each occasion the stress was carried well beyond the yield point, and was followed by immersion in boiling water for 10 minutes. The curves obtained are plotted in Fig. 5, the unit of strain being such that 12.85 units correspond to an angular distortion of 1 minute of arc. Curve I. shows the initial stress-strain curve, and curve II. represents the behaviour of the bar after 10 minutes' immersion in water at 100 degrees C. The remaining curves III. to IX. show the relation of stress to

* Muir loc. cit.

Fig. 5

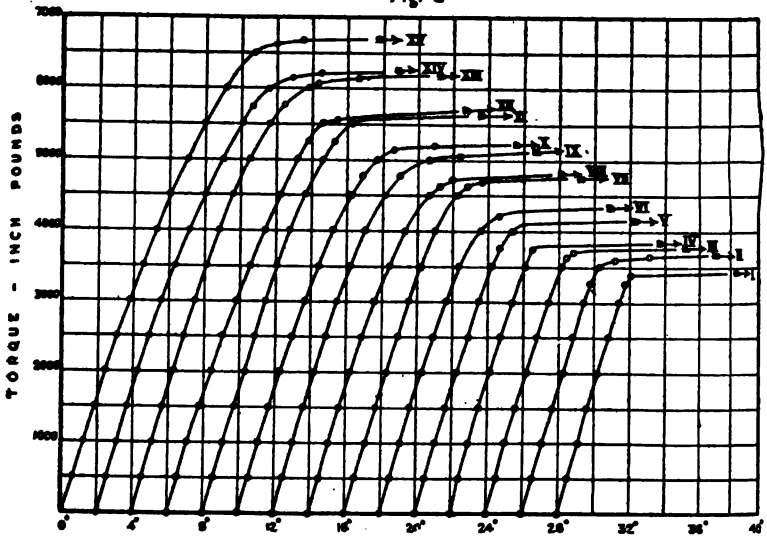



strain after each subsequent annealing in boiling water. After each stress and subsequent heating, the recovery was practically complete, and the yield point rose after each annealing. At the end of 9 separate twists the bar exhibited no change externally.

A test of a steel bar was also carried out by Messrs. Blanchard, Clement and Gagnon, Fourth Year students in the Civil Engineering Department. A turned steel bar of similar quality to the previous one was chosen, having a diameter of 0.757 inch, and a length under test of 8.00 inches. The strain measuring instrument was a simple graduated circle divided to 15 minutes, and readings could be estimated to 1 minute of arc. Each separate application of twisting moment was carried beyond the yield point,

and was followed by immersion in water at 50 degrees C. for 15 minutes. Even this low temperature was sufficient to cause a very effective recovery to the elastic condition, the recovery being in all cases accompanied by a rise in the yield point. The data ob-

Fig. 6




tained are exhibited in Fig. 6, and the history of the treatment of the bar is told by the curves marked I. to XV., I. being the initial and XV. the final curve of the series. Even at the end of the fifteenth twist, when the total permanent twist had reached .270 degrees, there seemed no probability of the bar fracturing.

The remarkable effect of a slight increase in temperature in restoring an overstrained bar to approximately its primitive condition may ultimately prove to be of considerable practical importance, as it affords a means of annealing, which, although not as perfect as that accomplished by the blacksmith, has the advantage that polished surfaces of iron and steel retain their polish and are not distorted by the process. In bridge structures and the like, exposed to repeated stresses and corresponding fatigue of the metal, the suggestion that this fatigue may be partially counterbalanced by the heat of a summer's day may not be so improbable as it at first sight seems to be.

## THE EFFECT OF LOW TEMPERATURE UPON THE RECOVERY FROM OVERSTRAIN.

It is interesting to determine what action low temperature, such as that afforded by a Canadian winter, has upon the behaviour of steel after overstrain, and in order to contrast this with recovery at

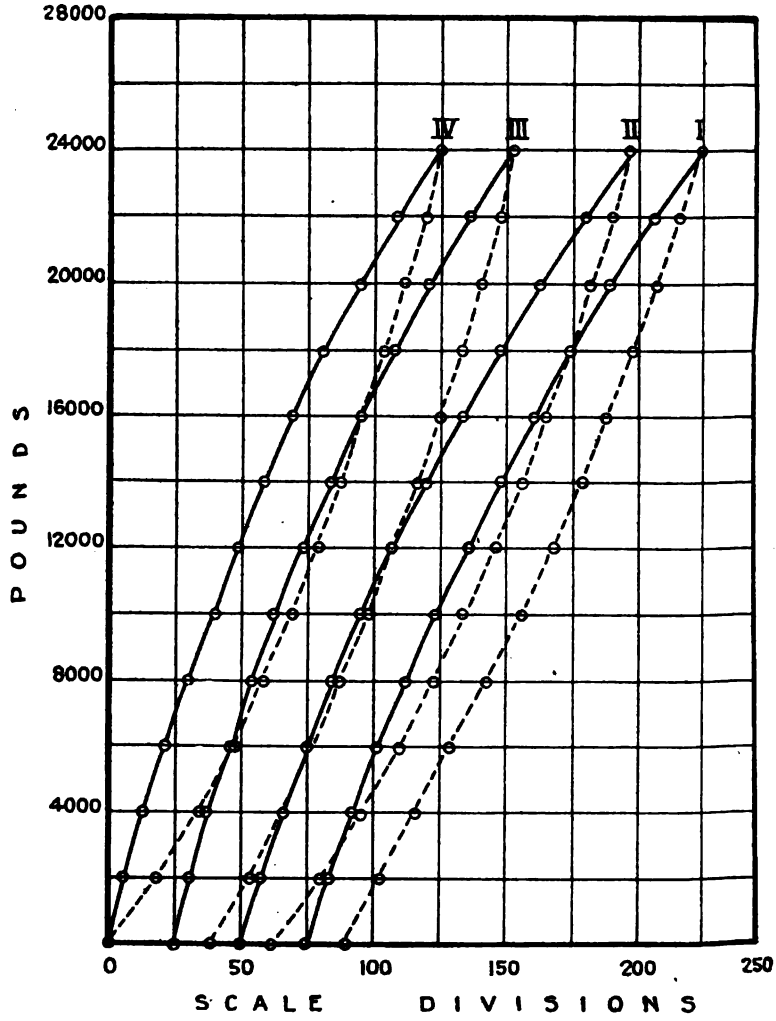
the temperature of the Laboratory, viz., about 68 degrees Fahr., a specimen was taken similar to the one described with reference to Fig. II. and the accompanying tables. The bar was 2.07 inches in breadth, and had a mean thickness of 0.378 inch, the length under test being as before, 8 inches. The specimen was first permanently stretched in exactly the same manner as in the previous case, and a comparison of the appended table with that on page 4 shows a very close agreement.

December 15th, 1900.

| Load, pounds. | Reading. | Δ |
|---|---|---|
| 0 | 4 0 | |
| | | - 34 |
| 2,000 | 434 | |
| | | —36 |
| 4,000 | 470 | |
| | | —36 |
| 6,000 | 506 | |
| | | —35 |
| 8,000 | 541 | |
| | | —33 |
| 10,000 | 574 | |
| | | —34 |
| 12,000 | 608 | |
| | | —34 |
| 14,000 | 642 | |
| | | —35 |
| 16,000 | 677 | |
| | | —34 |
| 18,000 | 711 | |
| | | —33 |
| 20,000 | 744 | |
| | | —42 |
| 22,000 | 786 | |
| | | —82 |
| 24,000 | 868 | |
| 26,000 | | 8 08″ |
| 27,000 | | 8.12″ |
| 27,500 | | 8 16″ |
| 0 | | 8 155″ |

The specimen was then exposed to the outside temperature for 3 days, during which the temperature varied from a minimum of —2.5 degrees Fahr. to a maximum of 11.2 degrees Fahr., and it was then subjected to a cycle of stress ranging from zero to 24,000 pounds. The results are recorded in Fig. VII., Curve I., and a comparison with Fig. II shows it to have a fair agreement with Curve III. of that figure. The bar was again exposed to the outside temperature, which had a mean value during the next four days of 18.52 degrees Fahr., a minimum value of —2.5 Fahr., and a maximum value of 31.72

Fig. 7




degrees Fahr. A second test, on December 18th, showed very little change in the condition of the bar, as will be seen by comparing the figures in Table III. with those previously obtained, and the bar was therefore given a long rest through the remainder of December and the whole of January. The mean temperature for the remaining days of December was 25.64 degrees Fahr., the highest 39 degrees Fahr., and lowest 0.6 degree

Fahr. During January the mean temperature was 12.75 degrees Fahr., the lowest temperature recorded being —16.7 degrees Fahr., and the highest 39.6 degrees Fahr. The bar was tested again on February 1st, see Table III. and Curve III., Fig. VII., and, as will be seen, it exhibited some slight indications of recovery, but very small in comparison with the recovery which would have been effected by the ordinary indoor temperature. A final test was made on Feb. 26, the mean temperature during the interval being 12.46 degrees Fahr., with a minimum of —4.0 degrees Fahr., and a maximum of 28.4 degrees Fahr. The recovery during this period was practically nil. It appears, therefore, that low temperatures tend to retard the recovery of steel, which has been subjected to overstrain, in quite marked degree.

TABLE III.

| Load Lbs. | Dec. 18, 1900. I. | | Dec. 22nd. II. | | Feb'y 1st. III. | | Feb'y 26th. IV. | |
|---|---|---|---|---|---|---|---|---|
| | Reading. | Δ | Reading. | Δ | Reading. | Δ | Reading. | Δ |
| 0 | 400 | -34 | 400 | -34 | 400 | -32 | 400 | -31 |
| 2,000 | 434 | -35 | 434 | -33 | 432 | -32 | 431 | -32 |
| 4,000 | 469 | -33 | 467 | -34 | 464 | -34 | 463 | -33 |
| 6,000 | 502 | -36 | 502 | -34 | 498 | -33 | 496 | -34 |
| 8,000 | 538 | -35 | 536 | -35 | 531 | -31 | 530 | -34 |
| 10,000 | 573 | -38 | 571 | -36 | 562 | -37 | 564 | -34 |
| 12,000 | 611 | -37 | 607 | -38 | 599 | -34 | 598 | -34 |
| 14,000 | 648 | -37 | 645 | -38 | 633 | -37 | 633 | -38 |
| 16,000 | 685 | -39 | 683 | -39 | 670 | -38 | 671 | -35 |
| 18,000 | 724 | -39 | 722 | -40 | 708 | -38 | 706 | -38 |
| 20,000 | 763 | -42 | 762 | -42 | 746 | -39 | 744 | -39 |
| 22,000 | 805 | -44 | 804 | -43 | 785 | -43 | 783 | -42 |
| 24,000 | 849 | -33 | 847 | -32 | 828 | -31 | 825 | -31 |
| 22.000 | 816 | -34 | 815 | -32 | 797 | -32 | 794 | -32 |
| 20,000 | 782 | -33 | 783 | -33 | 765 | -32 | 762 | -33 |
| 18,000 | 749 | -36 | 750 | -34 | 733 | -34 | 729 | -33 |
| 16,000 | 713 | -34 | 716 | -34 | 699 | -32 | 696 | -33 |
| 14,000 | 679 | -37 | 682 | -36 | 667 | -35 | 663 | -34 |
| 12,000 | 642 | -37 | 646 | -38 | 632 | -33 | 629 | -35 |
| 10,000 | 605 | -38 | 608 | -35 | 599 | -38 | 594 | -36 |
| 8,000 | 567 | -38 | 573 | -38 | 561 | -34 | 558 | -36 |
| 6,000 | 529 | -38 | 535 | -39 | 527 | -36 | 522 | -37 |
| 4,000 | 491 | -38 | 496 | -40 | 491 | -37 | 475 | -35 |
| 2,000 | 453 | -39 | 456 | -42 | 454 | -41 | 440 | -37 |
| 0 | 414 | | 412 | | 413 | | 403 | |

In conclusion, the author desires to thank Professor Bovey, Past-President of the Institution, who placed the Testing Laboratory of McGill University at his disposal; Prof. McLeod, Secretary of the Society, who kindly supplied the information regarding the temperature variations from the McGill Observatory records, and Messrs. Blanchard, Clement and Gagnon, who made all the observations recorded on Fig. 6.

Thursday, 23rd May.

MR. G. H. DUGGAN, Vice-President, in the Chair.

The following donation to the Library was reported:—
"The Progress of the Century," from Thomas C. Clarke.

**Paper No. 159.**

## DISCHARGE OF SEWAGE FROM TORONTO SEWERS IN 1900.

BY CECIL B. SMITH, M. CAN. SOC. C. E.

During March, April and May, 1891, the discharge of sewage from Toronto sewers was measured by Mr. C. H. Rust, M. Can. Soc. C.E., then engineer of sewers, and the results placed before the Society in January, 1893 (Paper 71). Since that time the city has taken up the question of sewage disposal in a cursory manner only. Nothing but preliminary work has been yet entered into, which has roughly consisted of levels and projects for various trunk sewers, etc., made at various times in past years, and recently cross-sections for filter beds and the measurement of the dry weather flow of sewage, which were carried out by the writer during the winter of 1899-1900.

The determination of the dry-weather flow of sewage was made with a view of confirming previous records, amplifying them, and establishing recent, accurate data on which to base the designs for interceptors, trunk sewers, and disposal works. The gauging was undertaken with considerable care, and carried out as accurately as possible; the weirs which had been previously used were reconstructed, or repaired, others made, and, to insure accuracy, a steel edge was added to the crest of the rectangular wiers in thin partition, the weir-depths being taken by hook gauges.

The setting and checking of weirs and recording of depths were in direct charge of G. F. Hanning, A. M. Can. Soc. C. E., while the work was done by four of the most trusted and careful employees of the sewer department, and it can be relied on for accuracy.

The records were taken, at each sewer, as will be seen by the charts attached, at each hour of the day and night for five or six days, but only during periods when there was no rain or thaw. The work extended from Jan. 9 to March 13, 1900.

The method of procedure was briefly as follows:—Weirs were

placed just at the upper sides of the manholes, wedged against struts and carefully puddled, platforms were built in the manholes from which to take the observations, after which the hook gauges were fixed about two feet up stream from the weirs, and adjusted in a slot or groove alongside a scale graduated to ⅛″. So that the zero of the scale and the arrow on the hook gauge arm coincided when the point of the hook was level with the crest of the weir, the weir-depths could thus be taken off by inspection, which lessened any chance of error. A lantern was suspended in the sewer and a little practice enabled the readings to be taken with great accuracy, as proven by frequent check readings taken by Mr. Hanning and the writer.

The detailed dimensions of sewers and weirs, and the attachments for gauging are shown on Plate I.

Before commencing computations, it was seen to be easier to draw up a table of discharges for various widths and depths to work from, and this was done; this table is appended hereto, because it can be applied to any width of weir by using fixed increments, and may therefore be of use to members of the Society. Several authorities were consulted, but the Francis formula, with end contractions, was adopted as being probably most accurate (see table of discharges for the formula) and in using the table as prepared, a correction for velocity of approach was made by the proper increase in depth of water over the crest of weir, for each sewer and depth. The results given for dry-weather flow (Table A) show that since 1891, the dry-weather flow of sewage has increased as follows:—

| | 1891 | 1900. |
|---|---|---|
| Gross Dry Weather Flow, Imp. galls. per day | 13,421,672 | 20,745,473 |
| Net " " " " " " " | 12,972,344 | 19,847,551 |

The net flow being less than the gross flow by the amount of sewage or water received by the city sewers from other municipalities, which amount was gauged in 1900 and estimated for 1891. At these same periods the average water pumped by the Toronto Water Works was as follows:—

| | 1891 | 1900. |
|---|---|---|
| Water pumped, Imp. galls. per day.. .. .. .. | 14,976,000 | 21,786,500 |

which indicates that the same conditions prevail now as in 1891 i. e., that either the pumping records are in error (which is not likely), or else that there are 2,000,000 gallons or over per day of water pumped which does not reach the sewers, being in various ways evaporated, etc., and that there is therefore no great amount of ground water entering the sewers during the winter months.

The pumping records are furnished through the kindness of

Mr. C. L. Fellowes, Deputy City Engineer, and the quantities are based on accurate tests made in 1898, allowance being made for slippage.

Whether this allowance was sufficient or not is perhaps a debatable point.

A study of Table A. will show that : (1) The total area sewered is approximately 7,426 acres. The population of the city was (1899) by Assessment Rolls, 192,162, which, judging by previous records, should be increased 13 per cent. to give the actual population, or say, 217,100, which gives corresponding populations per acre of:

$$\begin{cases} 25\ \frac{4}{10} \text{ by assessment rolls.} \\ 28\ \frac{7}{10} \text{ by estimated population.} \end{cases}$$

although this varies from 9 $\frac{2}{10}$ per acre to 54 $\frac{3}{10}$ per acre on different sewer areas. The daily average flow per acre is 2,794 Imperial gallons, varying from 890 to 7,439 Imperial gallons in different districts. (2) The sewage per head per day varies from 49½ Imperial gallons in a thinly populated residential district, to 724 Imperial gallons in a manufacturing and office building district, but averages:

$$\text{OR} \begin{cases} 105\ \frac{2}{10} \text{ Imperial gallons based on Assessment Rolls.} \\ 93\ \frac{1}{10} \quad \text{" \quad " \quad " \quad " estimated population.} \end{cases}$$

and it is noticeable that all those small sewer systems, like Bay Street, Front Street, Eastern Ave. W., and Berkeley Street, which are in business districts, have large flows per head per day, owing to the small resident population, and to the liberal use of water for elevator work, and manufacturing. The next largest flows per head per day are on the Yonge, Church, Jarvis and Bathurst districts, all over 100 gallons per head per day.

These districts are partly business and partly residential, and being in the older portions of the city, have perhaps more than their share of leaky water fixtures, defective plumbing, etc. The remainder of the sewers mostly run at 80 to 90 gallons per head per day (as we must deduct county water from Rosedale and Garrison Creek sewers, making them each 90 gallons) based on Assessment population, or say 70 to 80 gallons per day for an estimated total population of 217,000. (3) Various calculations may be made for each sewer to show how long it would take for ¼, 1-3, ½, etc., of the daily flow to arrive at the outlets, but one calculation for the whole city will serve as an illustration. It will take to arrive at the outlets of all the sewers of the city:

5 $\frac{1}{6}$ hours for ¼ of the total daily sewage.
6 $\frac{9}{10}$ " " 1-3 " " " " "
10 $\frac{6}{10}$ " " 1-2 " " " " "

(4) A study of the curves of flow on Plates II. and III. for each

main sewer, and the curves of gross and net dry weather flow for the whole city along with the diagram of pumping records will bring out two points very sharply.

First that although the flow of sewage is, of course, 25 per cent. to 100 per cent. greater during the day than in the night, yet the flow during the "wee sma'" hours is very great, and making some allowance for ponding of sewage during the day, in sewers with solids deposited on the inverts, which ponding would all get away soon after midnight, we have the bald fact that from 1 a.m. to 5 a.m., the flow of sewage is at the rate of 15,000,000 to 16,000,000 gallons per day, with as before stated, no probability of any great amount of ground water to draw from.

In the second place, if we take the sewage flow and compare it with the pumping records from 1 a.m. to 5 a.m., the cause of this flow is evident, for, with reservoir closed off, the pumping during these hours was at the rate of over 15,000,000 gallons per day in order to keep up the pressure in the mains.

Now the writer does not desire to specifically point a moral or adorn a tale by trying to dwell at length on the subject of water waste prevention, as Toronto is no worse a sinner in this respect than are most cities of this continent, but it would appear that as there is very little water used in the middle of the night, some 12,000,000 to 15,000,000 Imperial gallons per day must be going to waste somewhere, most likely the loss is caused largely by the innumerable leaks, bad plumbing, etc., of interior fixtures and services. If this is the case, we must look toward proper plumbing laws, with proper legal enforcement, rigid, incessant inspection and supervision of the installation and repairs of plumbing work, and the throwing of the burden on the tenant and landlord both as the means for reducing the consumption to something in the neighbourhood of 30 to 40 Imperial gallons per head per day, which represents good practice in Great Britain, a country inhabited by a clean, industrious, and industrial people, and what is enough for them should suffice us here.

This subject of water waste prevention is always important, but particularly so, now that the day is fast approaching when the treatment and disposal of the Toronto sewage and that of many other Canadian cities and towns must become a subject to be grappled with and solved, and then it will be doubly essential that the amount of sewage to be treated shall be kept down to as small an amount as possible, as the treatment of 20,000,000 gallons per day of even dilute sewage is a problem, the magnitude of which only the initiated are likely to appreciate. At present, there are only a few sewage disposal plants of such a size, and of either the septic tank or contact bed types none of such a size, and it is probable that bacteriological treatment will be the method adopted, when the question is finally taken up for solution in Toronto unless it is delivered in a crude form at a distance from the city into deep water of Lake Ontario.

# DISCHARGE OF WATER OVER WEIRS.

## FRANCIS FORMULA.

$$G = 2.4953\left(l - \tfrac{1}{10}nd\right)d^{\frac{3}{2}}$$

| DEPTH OVER WEIR. | GALLONS PER MIN. FOR WEIRS OF FOLLOWING WIDTHS. | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 7" | 14" | 18" | 22" | 30" | 36" | 40" | 50½" |
| ½" | 0.77 | . | . | .... | . . . | . . | | . . . |
| ¾" | 2.17 | 4.35 | .... | . . . | .... | .... | . . . | . |
| 1" | 3.97 | 7.96 | 10.27 | . . | .... | .... | . . . | .... |
| 1¼" | 6.09 | 12.26 | 15.79 | 19.32 | .... | . . . | .... | .... |
| 1½" | 8.47 | 17.10 | 22.02 | 26.96 | 36.83 | .. | . . | . . |
| 1¾" | 11.11 | 22.45 | 28.93 | 35.41 | 48.36 | 56.10 | .... | .... |
| 2" | 13.89 | 28.17 | 36.41 | 44.57 | 60.91 | 73.19 | 81.37 | . . . |
| 2¼" | 16.97 | 34.43 | 44.42 | 54.40 | 74.36 | 89.33 | 99.31 | 123.45 |
| 2½" | 20.16 | 41.01 | 52.93 | 64.83 | 88.65 | 106.52 | 118.48 | 149.70 |
| 2¾" | 23.54 | 47.94 | 61.90 | 75.85 | 103.75 | 124.67 | 138.62 | 175.24 |
| 3" | 27.06 | 55.22 | 71.31 | 87.41 | 119.50 | 143.73 | 159.83 | 202.07 |
| 3¼" | 30.73 | 62.80 | 81.14 | 99.48 | 136.15 | 163.65 | 181.99 | 230.11 |
| 3½" | 34.51 | 70.69 | 91.36 | 112.04 | 153.39 | 184.40 | 205.17 | 259.44 |
| 3¾" | 38.41 | 78.86 | 101.94 | 125.06 | 171.26 | 205.94 | 229.05 | 289.71 |
| 4" | 42.44 | 87.29 | 112.87 | 138.54 | 189.79 | 228.38 | 253.86 | 321.13 |
| 4½" | 46.59 | 95.99 | 124.22 | 152.45 | 208.91 | 251.36 | 279.49 | 353.59 |
| 4¾" | 50.83 | 104.93 | 135.85 | 166.77 | 228.60 | 274.96 | 305.90 | 387.06 |
| 5" | 55.16 | 114.11 | 147.80 | 181.49 | 248.85 | 299.39 | 333.08 | 421.50 |
| 5¼" | 59.59 | 123.52 | 160.06 | 196.59 | 269.65 | 324.46 | 360.99 | 456.89 |
| 5½" | 64.11 | 133.16 | 172.61 | 212.07 | 290.98 | 350.16 | 389.61 | 493.18 |
| 5¾" | 68.73 | 143.00 | 185.45 | 227.92 | 312.82 | 376.47 | 418.92 | 530.33 |
| 6" | 73.41 | 153.05 | 198.57 | 244.12 | 335.15 | 403.40 | 448.92 | 568.36 |
| 6¼" | 78.15 | 163.30 | 211.96 | 260.62 | 357.95 | 430.91 | 479.57 | 607.29 |
| 6½" | 82.99 | 173.79 | 225.61 | 277.47 | 381.20 | 458.99 | 510.86 | 647.00 |
| 6¾" | 87.88 | 184.37 | 239.51 | 294.65 | 404.88 | 487.64 | 542.77 | 687.51 |
| 7" | 92.94 | 195.16 | 253.68 | 312.14 | 429.10 | 516.82 | 575.30 | 728.61 |
| 7¼" | 97.86 | 206.16 | 268.04 | 329.93 | 453.70 | 546.53 | 606.42 | 770.37 |
| 7½" | 102.92 | 217.39 | 282.66 | 348.02 | 478.73 | 576.76 | 642.12 | 812.66 |
| 7¾" | 108.07 | 228.62 | 297.31 | 366.40 | 504.18 | 607.51 | 676.40 | 857.23 |
| 8" | 113.25 | 240.10 | 312.18 | 384.06 | 530.02 | 638.75 | 711.23 | 901.49 |
| 8¼" | 118.49 | 251.72 | 327.26 | 404.00 | 556.27 | 670.47 | 746.61 | 946.47 |
| 8½" | 123.78 | 262.50 | 343.18 | 423.20 | 582.90 | 702.68 | 782.53 | 992.13 |
| 8¾" | 129.09 | 275.43 | 358.60 | 442.67 | 609.92 | 735.36 | 818.97 | 1038.49 |

| DEPTH OVER WEIR. | GALLONS PER MIN. FOR WEIRS OF FOLLOWING WIDTHS. | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 7" | 14" | 18" | 22" | 30" | 36" | 40" | 50½" |
| 9" | .... | 287.50 | 374.95 | 462.40 | 637.30 | 788.46 | 855.68 | 1085.49 |
| 9¼" | .... | 298.70 | 391.04 | 482.98 | 665.05 | 822.63 | 892.40 | 1132.16 |
| 9½" | .... | 312.04 | 407.33 | 502.60 | 693.16 | 856.66 | 931.26 | 1181.47 |
| 9¾" | .... | 324.51 | 423.79 | 522.07 | 721.62 | 870.56 | 969.61 | 1230.4 |
| 10" | .... | 337.11 | 440.44 | 543.77 | 750.43 | 905.42 | 1008.75 | 1280.0 |
| 10¼" | .... | 349.83 | 457.27 | 564.71 | 779.57 | 940.73 | 1048.15 | 1330.2 |
| 10½" | .... | 362.68 | 474.27 | 585.87 | 809.05 | 976.44 | 1088.0 | 1381.0 |
| 10¾" | .... | 375.64 | 491.44 | 607.25 | 838.86 | 1012.56 | 1128.4 | 1432.4 |
| 11" | .... | 388.72 | 508.78 | 628.85 | 868.98 | 1049.1 | 1169.2 | 1484.3 |
| 11¼" | .... | 401.90 | 526.28 | 650.66 | 899.42 | 1086.0 | 1210.4 | 1536.9 |
| 11½" | .... | 415.20 | 543.94 | 672.69 | 930.17 | 1123.3 | 1252.0 | 1590.0 |
| 11¾" | .... | 428.60 | 561.76 | 694.92 | 961.23 | 1161.0 | 1294.1 | 1643.7 |
| 12" | .... | 442.11 | 579.73 | 717.35 | 992.59 | 1199.1 | 1336.6 | 1697.9 |
| 12¼" | .... | 455.72 | 597.85 | 739.98 | 1024.2 | 1237.5 | 1379.6 | 1752.7 |
| 12½" | .... | 469.42 | 616.11 | 762.81 | 1056.2 | 1276.2 | 1422.9 | 1808.0 |
| 12¾" | .... | 483.22 | 634.52 | 785.82 | 1088.4 | 1315.4 | 1466.7 | 1863.8 |
| 13" | . | 497.11 | 653.07 | 809.02 | 1120.9 | 1354.9 | 1510.8 | 1920.2 |
| 13¼" | .... | 511.10 | 671.75 | 832.41 | 1153.7 | 1394.7 | 1555.4 | 1977.1 |
| 13½" | .... | 525.17 | 690.57 | 855.98 | 1186.8 | 1434.9 | 1600.3 | 2034.5 |
| 13¾" | .... | 539.32 | 709.53 | 879.73 | 1220.1 | 1475.4 | 1645.6 | 2092.4 |
| 14" | .... | 553.56 | 728.61 | 903.65 | 1253.7 | 1516.2 | 1691.3 | 2150.8 |
| 14¼" | .... | 567.88 | 747.81 | 927.74 | 1287.6 | 1557.5 | 1737.4 | 2209.7 |
| 14½" | .... | 582.29 | 767.13 | 952.00 | 1321.7 | 1599.0 | 1783.8 | 2269.1 |
| 14¾" | .... | 596.77 | 786.60 | 976.42 | 1356.1 | 1640.8 | 1830.7 | 2329.0 |
| 15" | .... | 611.38 | 806.17 | 1001.01 | 1390.7 | 1682.9 | 1877.8 | 2389.3 |
| 15¼" | .... | 625.96 | 825.86 | 1025.8 | 1425.6 | 1725.4 | 1925.3 | 2450.1 |
| 15½" | .... | 640.66 | 845.68 | 1050.7 | 1460.7 | 1768.2 | 1972.2 | 2511.4 |
| 15¾" | .... | 655.43 | 865.56 | 1075.7 | 1496.0 | 1811.3 | 2021.5 | 2573.1 |
| 16" | .... | 670.26 | 885.61 | 1100.9 | 1531.6 | 1854.7 | 2070.0 | 2635.3 |
| 16¼" | .... | 685.16 | 905.74 | 1126.3 | 1567.5 | 1898.4 | 2118.9 | 2697.9 |
| 16½" | .... | 700.13 | 925.98 | 1151.8 | 1603.5 | 1942.3 | 2168.1 | 2761.0 |
| 16¾" | .... | 715.16 | 946.32 | 1177.5 | 1639.8 | 1986.6 | 2217.7 | 2824.5 |
| 17" | .. | 730.24 | 966.77 | 1203.3 | 1676.3 | 2031.1 | 2267.6 | 2888.5 |

| DEPTH OVER WEIR. | GALLONS PER MIN. FOR WEIRS OF FOLLOWING WIDTHS. | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 7" | 14" | 18" | 22" | 30" | 36" | 40" | 50½" |
| 17¼" | .... | 745.40 | 987.31 | 1229.2 | 1712.1 | 2076.0 | 2317.8 | 2952.9 |
| 17½" | .... | 760.60 | 1008.0 | 1255.3 | 1750.0 | 2121.0 | 2368.4 | 3017.7 |
| 17¾" | . . | 775.86 | 1028.7 | 1281.5 | 1787.2 | 2166.4 | 2419.2 | 3082.9 |
| 18" | . . | 791.18 | 1049.5 | 1307.6 | 1824.6 | 2212.1 | 2470.4 | 3148.5 |
| 18¼" | ... | 806.54 | 1070.4 | 1334.3 | 1862.2 | 2258.0 | 2521.9 | 3214.6 |
| 18½" | .... | 821.95 | 1091.4 | 1360.9 | 1899.9 | 2304.3 | 2573.7 | 3281.1 |
| 18¾" | .... | 837.42 | 1112.5 | 1387.7 | 1937.9 | 2350.6 | 2625.7 | 3347.9 |
| 19" | ... | 852.93 | 1133.7 | 1414.5 | 1976.1 | 2397.3 | 2678.1 | 3415.2 |
| 19¼" | .. | 868.48 | 1155.0 | 1441.5 | 2014.5 | 2444.6 | 2730.8 | 3482.9 |
| 19½" | . . | 884.08 | 1176.4 | 1468.6 | 2053.1 | 2491.5 | 2783.8 | 3551.0 |
| 19¾" | ... | 899.73 | 1197.8 | 1495.8 | 2091.9 | 2539.0 | 2837.0 | 3619.4 |
| 20" | .... | 915.41 | 1219.3 | 1523.1 | 2130.9 | 2586.7 | 2890.6 | 3688.2 |
| 20¼" | . | 931.14 | 1240.9 | 1550.6 | 2170.1 | 2634.7 | 2944.4 | 3757.5 |
| 20½" | .... | 946.90 | 1262.5 | 1578.2 | 2209.4 | 2682.9 | 2998.5 | 3827.1 |
| 20¾" | ... | . | 1284.3 | 1605.8 | 2249.0 | 2731.5 | 3053.9 | 3897.0 |
| 21" | ... | . . | 1306.1 | 1633.5 | 2288.7 | 2780.0 | 3107.6 | 3967.4 |
| 21¼" | . . | . | 1328.0 | 1661.5 | 2328.6 | 2828.9 | 3162.5 | 4038.1 |
| 21½" | ... | . . | 1349.9 | 1689.5 | 2368.7 | 2878.1 | 3217.7 | 4109.2 |
| 21¾" | ... | . . | 1371.9 | 1717.6 | 2409.0 | 2927.5 | 3273.2 | 4180.6 |
| 22" | ... | . . | 1394.0 | 1745.8 | 2449.4 | 2977.1 | 3328.9 | 4252.4 |
| 22¼" | ... | . | 1416.2 | 1774.1 | 2490.0 | 3027.0 | 3385.0 | 4324.6 |
| 22½" | .... | . | 1438.5 | 1802.5 | 2530.8 | 3077.1 | 3441.2 | 4397.0 |
| 22¾" | . | . . | . . | 1831.0 | 2571.7 | 3127.3 | 3497.7 | 4469.7 |
| 23" | . | . | . . . | 1859.6 | 2612.8 | 3177.8 | 3554.4 | 4543.0 |
| 23¼" | ... | . . | . . | 1888.2 | 2654.1 | 3228.5 | 3611.4 | 4616.6 |
| 23½" | ... | . . | . . . | 1917.1 | 2695.5 | 3279.4 | 3668.7 | 4690.5 |
| 23¾" | ... | . . | . | 1945.9 | 2737.1 | 3330.6 | 3726.2 | 4764.7 |
| 24" | ... | . . . | . . | 1974.8 | 2778.9 | 3381.9 | 3783.9 | 4839.2 |
| 24¼" | ... | . . | . . . . | 2003.9 | 2820.8 | 3433.5 | 3841.9 | 4914.1 |
| 24½" | . | . . | . . . | 2033.0 | 2862.9 | 3485.2 | 3900.2 | 4989.3 |
| 24¾" | . . | . . . | . | . . . | 2905.1 | 3537.2 | 3958.6 | 5064.8 |
| 25" | . . | . . . | . . . | . . . . | 2947.5 | 3589.4 | 4017.2 | 5140.7 |
| 25¼" | . | . . | . . | . . . | 2990.0 | 3641.7 | 4076.2 | 5216.8 |

* Gallons per min. l = Length of weir in inches. n = Number of end contractions of weir. d = Depth of water flowing over weir.

## DISCUSSION.

Mr. S. Howard remarked that he had carefully read Mr. Smith's paper on the discharge of sewage from Toronto sewers in 1900, the measurements having been taken in order to supply necessary information for disposal works. The method employed seemed to have been well studied, great care having been taken in order to arrive at the discharge of sewage during dry weather, and the results should be very useful to Engineers designing a system of sewerage, especially on the separate system plan. He was unable to discuss the paper proper, as it did not, to his mind, admit of much discussion. The purpose of the gauging was to find out the dry flow of sewage, which had been carefully and scientifically done and showed what a difference there was in the supply of water in different sections of the City, ranging from 49 gallons to 328 and 724 gallons per head, the latter being from districts containing large offices and factories. This information was of more use to engineers designing a water supply system than it was to those designing sewerage works, as it showed what a serious amount of water ran to waste at hours when little was being used, caused either by householders leaving taps running or by leaky joints in mains, and defective plumbing, which should be carefully looked into and rectified, if possible. He agreed with the author that 20,000,000 gals. of diluted sewage was a tremendous amount to handle and treat so that the effluent would not contaminate the lake. The formulae prepared by Mr. Francis, based on experiments carried on at Lowell, was about the most accurate one used in practice, and differed very little from those of Bazin and Herschel, and the labour performed in the calculations must have been prodigious. However, the reading of the paper, and the information gained, naturally lead to other topics, and his ideas resolved themselves into the question of the carrying away of storm waters, and the proper data to work upon in designing a sewerage system. Enough careful thought was not given to this question. A city or town should be laid out in blocks, with contours, the trunk sewers put on, the areas calculated (taking into consideration the slope of the ground, the nature of the surface, and the proper rainfall to be allowed per hour) and the sewers properly figured out as to size so as to be able to take this water. Rule of thumb was too often used without carefully going into future requirements, and the fact that the areas would be likely all paved in time.

In a combined system of sewers, the sewage proper was a very small item, the storm waters being the large quantity which had to be seriously considered. Taking the Toronto sewers running

full, the cubic feet of capacity per second was 1,765, whereas the sewage from the present population was only 39 cub ft. per second, or 1-45 of the total amount. This with an increased population of 50 per cent. would be increased to 60 cubic feet or say 1-30 of the whole quantity discharged. As mentioned above, the total capacity of all the sewers for the 24 hours was 1,765 cubic feet, and taking off 39 cubic feet of sewage, room was left for 1,726 cubic feet per second of storm water, which for 7,426 acres drained allowed for the discharge of .5 of one half inch per hour of rain or thaw. Increasing the population 50 per cent, bringing the total to 288,000 persons at 100 gallons per head per day, the sewage would amount to 60 cubic feet per second, leaving 1,705 cubic feet for storm waters, or .45 of ½ inch of rain per hour, but with this increased population if a heavy storm should occur at 10 a.m., being the hour at which the greatest amount of sewage is discharged, the sewers would only take .45 of one-half inch of rain per hour. He had taken about half a dozen of the sewers in order to arrive at the amount of storm water which each was capable of handling, but not knowing the slopes of the ground nor the nature of the pavements and surroundings, in order to calculate closely the amount of run off of water, the results were only approximate. We were often visited with very heavy rainstorms which were nearly always preceded by days of light rain, making the surface of the ground almost impervious. To design a system with the heaviest rainfall would be absurd as the sewers would be too large, and too costly. The question would then arise how much rainfall per hour should be allowed? Most systems were only able to contend against a percentage of one-half inch per hour. The sewers of the City of Montreal had been calculated upon this basis, which, in his opinion, was too small; to strike an average one inch should be allowed for. Heavy rains only lasted a short time and no great damage could be done either to cellars or streets, if the figure of one inch were the standard. Taking the Garrison Creek sewer of a diameter of eight feet and draining 2,112 acres, the discharge running full was equal to 405 cubic feet per second. whereas the sewage only amounted to 9 cubic ft., leaving 396 cubic feet per second for storm waters capable of taking .40 of one half inch of rain per hour.

The Yonge Street sewer, drainage 565 acres, had a capacity of 87 cubic feet of water per second, the sewage proper being only 7 cubic feet, leaving 80 cubic feet for storm water. This would take from the area drained .25 of one half inch per hour.

The Bay Street sewer, which had the largest amount of gallons per head, was capable of taking 29 cubic feet per second whereas the storm waters, which, with a drainage area of 29 acres, would allow of .6 of 1.66 inches of rain per hour.

Parliament Street sewer drained 240 acres, but the population was the heaviest on the whole district, and its capacity was 84 cubic feet per second, and the sewage from this population gave only 2.33 cubic feet leaving 81.66 for storm waters, equalling .66 of ½ inch of rain per hour.

Front Street sewer drained 31 acres, and although the population was small, still the water supply was heavy, being 328½ gallons per head. This was the second highest on the list, Bay Street district being the highest, and the total capacity was 13 cubic feet per second, of which the sewage was only .43 cubic feet, leaving for storm waters 12.57 cubic feet or .4 of a cubic foot per second per acre, equal to .60 of .66 of an inch of rain per hour, or say .5 of .8 of an inch per hour.

The Rosedale Creek sewer drained 758 acres, and had a capacity of 216 cubic feet per second, the sewage being only 2 cubic feet. The population was very light, and if it should be trebled there would still be 210 cubic feet per second available for storm waters, equalling .5 of .60 of an inch per hour.

The last on the list was the sewer on Lake Street, which had a drainage area of 10 acres. The sewer running full had a capacity of 30 cubic feet per second. The sewage, with the present population was almost nil, but allowing say 400 persons for this area, using 100 gallons per head per day, the sewage would only amount to .70 of a cubic foot per second, so that the sewer would take .5 of 5¾ inches of rain per hour. From a casual glance he thought that one would imagine this sewer to be altogether out of proportion to the others, but the presumption would be that there were overflows from higher sewers discharged into it.

The Author, in reply, observed that Mr. Stuart Howard's discussion was on two points: (1) The difficulty and expense of sewage treatment when such a large amount of wasted water must be treated. (2) The proper capacity of sewers in a combined system.

Regarding the first matter, conditions seemed hopeless in America when such quantities as 200 to 300 gallons per head per day were used in several cities, while in Great Britain, with apparently greater industrial demands on a water supply 50 to 60 gallons per head per day was found sufficient. When our people and City Councils were educated to the consistent use of meters and rigid, thorough, real, inspection of water fixtures in houses and street mains was carried out, and when laws governing the use and abuse of water were framed and enforced we might hope for marked improvement, but not before.

In Toronto, of recent years, all lateral sewers had been designed to take care of 1½ inches rainfall per hour, since a decision was rendered in the courts, fixing this as a limit beyond which munici-

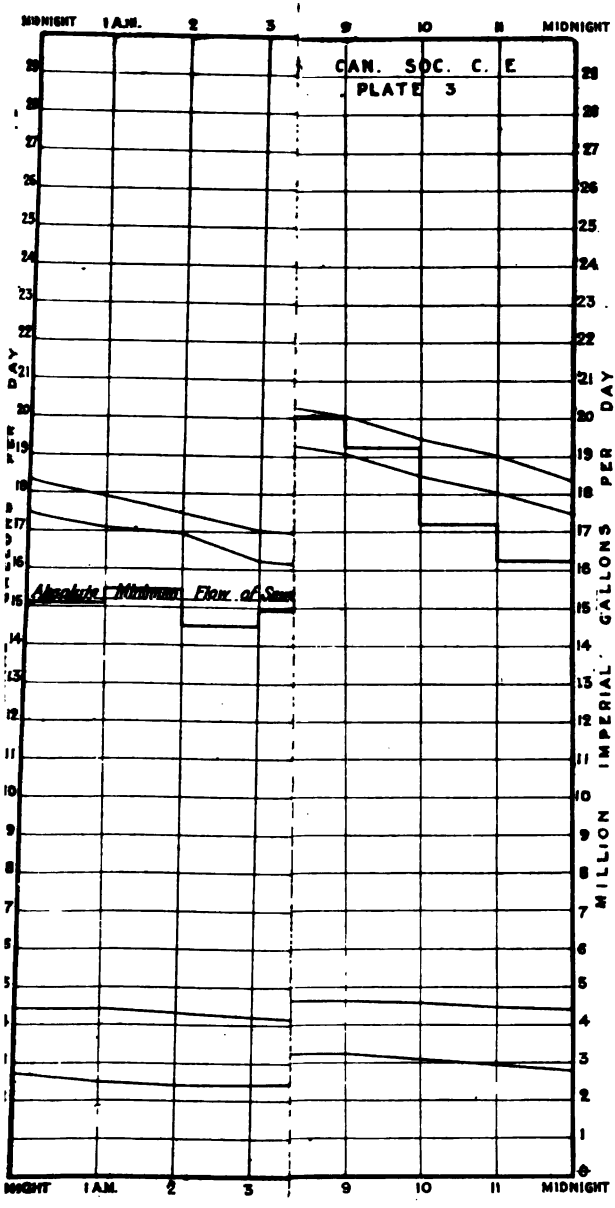
pal corporations would not be considered liable. Previous to this, 1 inch rainfall per hour had been the basis of calculation. This, of course, applied only to laterals and secondary sewers. Trunk sewers draining a large area were not designed of corresponding capacity as a storm need not be expected to be of this severity and at the same time to cover a large territory and continue for an hour without intermission.

The formula generally used in Toronto in proportioning the flow of storm water in the sewers was that given in Trautwine, known as the Burkli-Ziegler formula.

Cubic feet per sec. per acre reaching sewer = coefficient × average cub. feet of rainfall per sec. per acre during heaviest fall ×

$$\sqrt{\frac{\text{av. slope of ground in feet per 1000 ft.}}{\text{number of acres drained}}}$$

The Lake Street sewer referred to by Mr. Howard was designed to be the trunk sewer for a district not yet sewered; this accounted for its apparently excessive size. Whereas the Queen Street sewer shown as emptying by way of the Leslie Street sewer, had numerous overflows in to Logan, Morse and Carlaw Street sewers, etc.


CAN. SOC. C. E.
PLATE 3
MIDNIGHT
1 A.M.
2
3
9
10
11
MIDNIGHT
MILLION IMPERIAL GALLONS PER DAY
Absolute Minimum Flow of Sewer

CA

of A

M.

776
376
320
000
608
920
984
744
088
936
867
528
971
559
325
481
278
731
697
262
531
331
97
48
49

26
68

94

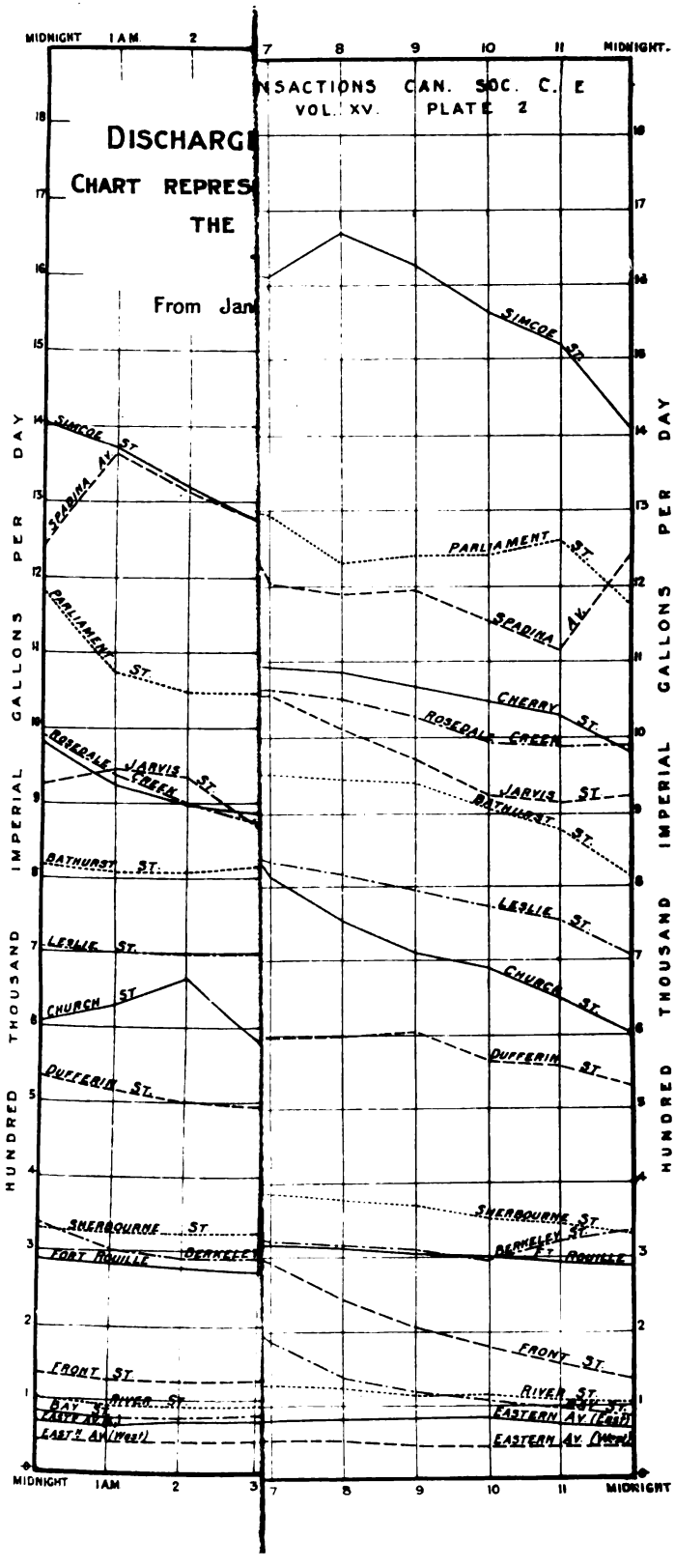
# CANADA TABLE A.

*of Approach Allowed*

| I. | 3 P.M. | 4 P. M | y flow acre. | Sewer Capacity Running full. | Dry Flow. Capacity. |
|---|---|---|---|---|---|
| 776 | 329,832 | 321, | 929 | 18,630,000 | $\frac{1}{61}$ |
| 376 | 644,400 | 621, | ,618 | 42,604 000 | $\frac{1}{74}$ |
| 320 | 5 168,592 | 5,074 | ,219 | 217,467,000 | $\frac{1}{48}$ |
| 000 | 1,029,024 | 992, | 291 | 39.970.000 | $\frac{1}{44}$ |
| 608 | 1,766,480 | 1.603, | 213 | 95.276.000 | $\frac{1}{65}$ |
| 920 | 1,801,440 | 1,697, | 721 | 61,236,000 | $\frac{1}{39}$ |
| 984 | 348,480 | 331, | 987 | 15,480,000 | $\frac{1}{78}$ |
| 744 | 3,994,344 | 3,924, | 844 | 45,864,000 | $\frac{1}{14}$ |
| 088 | 1,169,957 | 1,095, | 500 | 37,800,000 | $\frac{1}{43}$ |
| 936 | 1,125,000 | 1,066, | 868 | 62,838,000 | $\frac{1}{24}$ |
| 867 | 391,867 | 370. | 211 | 44,100,000 | $\frac{1}{119}$ |
| 528 | 422,496 | 414. | ,549 | 15,687.000 | $\frac{1}{43}$ |
| 971 | 1,385,971 | 1,371. | 243 | 45,144,000 | $\frac{1}{38}$ |
| 559 | 1,268,460 | 1,246. | 394 | 21,906,000 | $\frac{1}{30}$ |
| 325 | 335,765 | 321. | 439 | 6,948,000 | $\frac{1}{30}$ |
| 481 | 67.090 | 94, | 329 | 2,736,000 | $\frac{1}{40}$ |
| 278 | 133,214 | 123, | 754 | 14.346,000 | $\frac{1}{124}$ |
| 731 | 1,228,277 | 1,147, | 389 | 116,469,000 | $\frac{1}{118}$ |
| 697 | 86,961 | 86, | 736 | 9,171,000 | $\frac{1}{134}$ |
| 262 | 14,184 | 14. | 941 | 2,910,000 | $\frac{1}{325}$ |
| 531 | 25,531 | 25. | 380 | 3,207,000 | $\frac{1}{105}$ |
| 331 | 35,107 | 37. | 323 | 2,910,000 | $\frac{1}{98}$ |
| 973 | 51.156 | 53, | 419 | 3,486,000 | $\frac{1}{80}$ |
| 483 | 892,483 | 861, | 891 | 7,650,000 | $\frac{1}{9}$ |
| 491 | 11,491 | 12, | 890 | 16.290,000 | $\frac{1}{1810}$ |
| 260 | 22,727,602 | 22,910, | ..... | ..... | ..... |
| 685 | 369,466 | 356, | ..... | ..... | ..... |
| 945 | 24,097,068 | 23,267, | ,794 | ..... | ..... |
| ,399 | 497,520 | 495, | ..... | ..... | ..... |
| ,188 | 64,188 | 64, | ..... | ..... | ..... |
| 712 | 33,696 | 51, | ..... | ..... | ..... |
| 779 | 283,406 | 479 | ..... | ..... | ..... |
| 078 | 878,810 | 1.091, | ..... | ..... | ..... |
| 867 | 23,218.258 | 22,175, | .673 | ..... | ..... |

: L—Width of Weir.
d—Depth of Water f
n—Number of end c

Engineer,
, MARCH, 1900.


DISCHARGE
CHART REPRES
THE
From Jan
MIDNIGHT
1 A.M.
2
7
8
9
10
11
MIDNIGHT.
HUNDRED THOUSAND IMPERIAL GALLONS PER DAY
SIMCOE ST.
SPADINA AV.
PARLIAMENT ST.
ROSEDALE CREEK
JARVIS ST.
BATHURST ST.
LESLIE ST.
CHURCH ST.
DUFFERIN ST.
SHERBOURNE ST.
BERKELEY ST.
FORT ROUILLE
FRONT ST.
RIVER ST.
BAY ST.
EASTERN AV. (East)
EASTERN AV. (West)
CHERRY ST.

NOV 14 1902

VOL. XV. 1901. PART II.

# TRANSACTIONS

OF THE

# Canadian Society of Civil Engineers

OCTOBER TO DECEMBER, 1901.

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OF

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VOL. XV., PART II.

OCTOBER TO DECEMBER,
1901.

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## INSTRUCTIONS FOR PREPARING PAPERS, ETC.

In writing papers, or discussions on papers, the use of the first person should be avoided.

They should be legibly written on foolscap paper, on one side only, with a margin on the left side.

Illustrations, when necessary, should be drawn on the dull side of tracing linen to as small a scale as is consistent with distinctness. They should not be more than 10 inches in height, and *in no case* should any one figure exceed this height. Black ink only should be used, and all lines, lettering, etc., must be clear and distinct.

When necessary to illustrate a paper for reading, diagrams must be furnished. These must be bold, distinct and clearly visible in detail for a distance of thirty feet.

Papers which have been read before other Societies, or have been published, cannot be read at meetings of this Society.

All communications must be forwarded to the Secretary of the Society, from whom any further information may be obtained.

The attention of Members is called to By-laws 46 and 47.

# CIVIL ENGINEERS.

## YEARS 1887 TO 1902.

| 1895. | 1896. | 1902. |
|---|---|---|
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| M. MURPHY. | CHAS. MACDO | H. DUGGAN. |
| H. WALLIS. | W. G. THOMP | W. BLACKWELL. |
| | | ARCEAU. |
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| | | IRWIN. |
| | | A.RHYS-ROBERTS. |
| WM. MCNAB | WM. MCNAB. | |

Thursday, 24th October.

G. H. DUGGAN, Vice-President, in the Chair.

The following donation to the Library was reported:—"The Engineer or Architect as the Arbitrator between the Employer and the Contractor," by Charles C. Gregory, M. Can. Soc. C.E.

**Paper No. 160.**

## ECONOMICS OF RAILWAY IMPROVEMENTS.

BY W. W. COLPITTS, M.Sc., A.M. CAN. SOC. C.E.

The question of the most advantageous methods of improving existing railway lines, and the effect of such improvements upon operating expenses, is one to which the attention of the railway engineer of the present day is frequently directed. The object of this paper is to outline methods by which the most economical results may be obtained in operation, and to define a basis upon which the yearly value of railway improvements may be quickly and accurately estimated; at the same time to epitomize a vast and varied subject into such brief space as to facilitate its ready application in practice.

The profession of Civil Engineering has been defined as "the art of making a dollar earn the most interest," a definition which in no branch of the profession should be more closely adhered to as a maxim than in railway engineering. Evidences are to be found on every hand of enterprises in which the financial aspect has been largely lost sight of by engineers who were desirous of building works of monumental excellence rather than of structural utility. The result, in many instances, has been the impoverishment of the company before the line could be equipped for economical operation, or even completed. The promoters of railway projects, on the other hand, in their zeal to reduce first cost, have, in many cases, constructed lines of such a character that the expense of operation has bankrupted enterprises that might otherwise have proved financially successful. Between these extremes there is always a happy mean which will give the largest return for the money invested, and upon the engineer's ability to discover the wisest middle course may depend the success of the venture. Generally speaking, the most economical line to adopt may be defined as that on which the sum of the operating expenses and the interest charges on the total expenditure are least.

Many existing lines, built under limited financial conditions, in the expectation of a sufficiently large traffic in the future to warrant radical improvements, have been greatly handicapped because of the failure of the promoters to recognize the paramount importance of adopting the lowest rate of grade which the country would afford. Unfortunately, engineers are prone to work to the extreme limit of grades and curves, even to the extent of adopting a maximum grade in localities where a minor gradient might as easily have been obtained. The result of such lack of foresight is that any attempt to improve the grades involves practically the reconstruction of the whole line. Until the traffic has developed sufficiently to justify such radical changes the line is operated under most unfavourable conditions, which in numerous instances has resulted in the appointment of a receiver.

However favorable the conditions under which a line is projected, there is but a small margin for error either during construction or in later improvements. Mistakes in alignment may be rectified, structures rebuilt and local sags removed, but the ruling grade of a line, once established, is the limiting feature which governs largely the expense of operation.

Many lines, and particularly trans-continental lines, have, largely by their own agencies, developed the traffic which they carry, and with little hope of immediate return have been built in the cheapest possible manner with heavy grades and sharp curvature.

Later on, with the development of traffic and the impetus of competition, the line is inadequate to the business, and means must be sought for increasing its facilities. Railways constructed under such conditions should seldom, if ever, be double-tracked without the intermediate step of reducing the grades. This is particularly true of lines with but little more traffic than can be handled economically and expeditiously. The convenience and rapidity with which trains can be operated on a double track line, and the sentimental bias of the public in their favor, have been incentives towards the double-tracking of many lines, when to have reduced the grades would have involved a much less expenditure of money. Viewed from a financial standpoint, the latter course will usually pay a greater percentage of interest on the investment, at the same time facilitating the economical handling of a larger volume of traffic than was previously possible.

In the case of lines with a heavy passenger or suburban traffic, the conditions are quite reversed. The question becomes one rather of the number of light trains which can be operated rapidly and in quick succession, and without danger of delay, than of the number of tons that can be handled in a train. To increase the number of tracks as the business justifies it is evidently the solution of the problem.

The cost of double-tracking will seldom be less than fifty per cent. of the first cost of construction; and in rough country, where it may be necessary to completely divert the second track, it will more often approach seventy-five per cent. Rails and fastenings, ties and ballast, will be approximately one hundred per cent. of the cost of such items in first construction. The cost of grading may vary from forty per cent. to sixty per cent. of the first cost, depending upon the topography of the country; the cost of bridging from seventy-five to ninety per cent. From this it is evident that the traffic of a line must be very considerable, and greatly congested, if the heavy expenditures thus entailed cannot be postponed for a number of years by a reduction of the grades. As an illustration of the advantages of our latter course, a reduction of a ruling gradient from 1 per cent. to 0.4 per cent. practically doubles the haulage capacity of locomotives and reduces the number of freight trains about fifty per cent., and the expense of operation about twenty-five per cent.—a course which, on lines with a moderate traffic, is usually warranted by the consequent reduction in operating expenses alone, aside from the advantages afforded for handling an increase of business.

Simultaneously with grade reductions, many other improvements may be effected without adding materially to the cost, at the same time placing the line in a fit condition to be double-tracked in the future. At no time is a better opportunity offered for putting in transition ends to curves which have not been previously eased. Objectionable features of alignment may be removed and curvature eliminated. In many cases it will be found possible to consolidate a number of heavy grades in one section, which may then be operated with comparative economy, but which the traffic would not warrant reducing, and thus full benefit may be derived from improvements in adjoining sections. It will usually be found more economical to operate heavy grades in the vicinity of divisional points with the assistance of the yard engines, reducing the grades on the remainder of the section; and at points where it would be necessary to construct several miles of new line, to avoid a short heavy grade, it will often be found more economical to double the train.

Little advantage is to be obtained in operation from improvements which are not calculated to increase the engine rating for the whole section. An excessive ruling gradient at one point, the necessity for stopping on a grade, or insufficient compensation for curvature on a ruling gradient, may appreciably limit the rating of the section. In order, therefore, to obtain the most ecocomical results in operation, improvements should be designed to make the rating uniform on all ruling gradients.

## DISTANCE.

The extent to which freight and passenger receipts will be affected by minor changes in distance depends largely upon the nature of the traffic, and upon the geographical position of the line in relation to its competitors. The relative proportions of the different classes of traffic vary greatly on different roads or on different parts of the same road, so that each case must be carefully studied with reference to the peculiar conditions which obtain on that particular section. It is evidently impossible in a general discussion of the subject, to obtain results which may be applicable in all cases. We may, however, define a basis upon which to proceed intelligently to estimate the effect of such changes upon freight and passenger revenue, as well as upon operating expenses, which latter we are able to determine with considerable accuracy. For this purpose, we divide the traffic into three distinct classes. Traffic of any other nature will, so far as this estimate is concerned, be found to belong to one or the other class:

CLASS 1.—Local and exchange traffic, non-competitive, between points on one line, or passing over two or more lines, but having no option as to route.

CLASS 2.—Local traffic, competitive, between points on one line, but having an option of two or more routes.

CLASS 3.—Through exchange traffic, competitive, passing over several lines, with an option of two or more routes.

Non-competitive traffic, Class 1, usually represents the major proportion of the total on all roads, and a shortening of the line will at once result in a direct loss of revenue. This loss is proportional to the amount by which the line is shortened, and may be obtained as follows: Multiply the total number of revenue tons and passengers passing over that portion of the line in a year, by the average revenue per ton, and per passenger mile, from such traffic, and by the number of miles by which the line is shortened.

The rates on local competitive freight traffic, Class 2, are usually arranged on a mileage basis, unless arbitrarily fixed on account of boat or other competition, with a more or less disregard of the actual distance traversed. In the latter case the revenue will be neither increased nor decreased by slight changes in the length of the line. In the case of local competitive passenger traffic, the rates are determined by the mileage of the shortest line. If based on the mileage of the home line, the revenue from this traffic may be materially reduced by shortening the line, which loss may be

estimated, as in the case of Class 1. If the home line is one of the longer routes, the receipts will not be affected by changes of distance.

The revenue from through exchange competitive traffic, Class 3, may be seriously reduced by shortening the line. The rates on such traffic, as in Class 2, are usually established in the case of freight with a more or less disregard of distance, and in the case of passenger by the mileage of the shortest line; but the division of the total between the lines participating in the business is in the proportion of the mileage of each, or on a *pro rata* per rate basis. It is evident, therefore, that so far as the class of traffic is concerned, if the line be shortened, its proportion of the revenue will be correspondingly decreased. Some lines, on account of physical advantages or for various reasons, may be given "differentials" or "reconstructive mileage" or "arbitraries" for terminal bridge transfer, or other expenses, in addition to their proportion of the remainder of the revenue. To estimate the loss which would result to this class of traffic by shortening the line, we first deduct the amount of the arbitraries from the total yearly revenue from such traffic, and if the receipts are pro-rated on a mileage basis we multiply the balance by the difference between the ratios of the home mileage to the total average mileage before and after the contemplated changes. If the revenue is apportioned on a *pro rata* per rate basis, this latter multiple is determined accordingly.

Summarizing the foregoing, from Classes 1 and 3, it is seen that a direct loss of revenue will result from shortening the line; the revenue from Class 2, which at most is but a small proportion of the total, will be affected injuriously or not at all, according as the home line is shorter or longer than its competitors. Thus it might appear that there is no credit side to eliminating distance other than would result from the saving in operating expenses. This, however, is true only of such traffic as has been actually obtained. The shortest line between competitive points is, other things being equal, in much the most advantageous position for securing the largest proportion of the traffic, and any improvements tending to shorten the longer line will prove decidedly beneficial in that respect, but once the business is secured the advantage is wholly in favour of the long haul.

To the ordinary passenger, time is the first consideration. Distance is largely a technicality which no more concerns him than the grades or curves of the line, and the quickest route to his destination is the most likely to receive his patronage. Whatever the character of the improvements, therefore, whether of the nature

of grade and curvature reductions or the elimination of distance, if they are such as will facilitate the running of faster trains, they will tend to increase the volume of traffic, and, consequently, the revenue.

The expenses of operation will be more or less affected by minor changes in distance which may be necessitated by grade reductions. If the agreements between the Company and its employees are not based on mileage, train wages will not usually be subject to modification because of a slight change in the length of the section. Track force and maintenance of way expenses will depend upon the extent of the change.

The following estimates of the effect of slight alterations in distance on operating expenses are based on Wellington's results, and are figured for an average train mile cost of 85 cents. Having determined the actual train mile cost on any particular section, these amounts should be adjusted proportionately:

| | If train wages are affected. | If train wages are not affected. |
|---|---|---|
| Changes aggregating from 0 to 2 miles | 33.7c. | 21.1c. |
| Changes aggregating from 2 to 15 miles | 43.7c. | 31.0c. |

The above figures, multiplied by the total number of trains each way in a year, give the value of one mile increase or decrease in distance. The number of trains to be used in this calculation will be that operated on the old line, or necessary to handle the same volume of traffic on the new line, according as the length of the section is reduced or increased by the improvements.

## REDUCTION OF RULING GRADE AT STATIONS.

At a station at which freight trains may be required to stop for water or other purposes, the engine rating may be materially reduced below what it otherwise would be, if at that point a grade of the same rate as the ruling grade were adopted. The rating for the whole section is thus limited to that which the locomotive can start on the ruling grade on which it is necessary to stop. It is the criterion of economical operation that, so far as possible, the engine be working with uniformity, and at its full capacity on all parts of a section. Yet instances are to be found, to a greater or less extent on all railways, in which the full power of the locomotive is demanded only at one or two points in an engine stage, where stations have been located on grades which have not been compensated for stopping. The consequent reduction in the rating for the whole section thereby greatly increases the expense of operation, but not usually to such an extent as to warrant the use of assistant

engines. The arbitrary method of compensating such grades two-tenths per cent. at stops is, as will be seen from the following, much too little:

FIG. I.

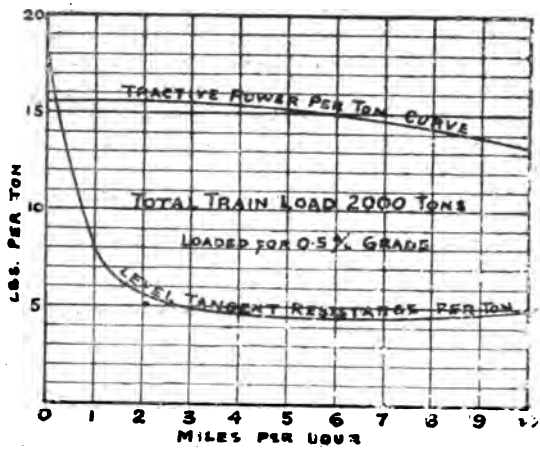



Figure 1 gives, in lbs. per ton, the train resistance curve and the traction power curve of a modern locomotive for speeds from 0 to 10 miles per hour. At the point of starting the resistance is much greater than at 5 or 7 miles per hour, when it reaches a minimum of about 4½ lbs. per ton. The tractive power, on the other hand, remains practically constant between these speed limits, Compound engines, by being thrown into simple, may increase their tractive power at low speeds, and thus possess a reserve force for starting or surmounting short heavy grades.

With the old form of link couplers trains could be started from rest on much heavier grades than is possible at present with automatic couplers. The slack between the cars was much greater, and the locomotive had acquired a considerable speed before the caboose began to move. The very great starting resistance was thus overcome in the forward part of the train when but a small proportion of the load was being acted upon.

Taking three inches as the average draw bar extension in freight cars, in a train loaded for a 0.5 per cent. grade, or say forty-one cars, there is 20 feet slack. In that distance, the engine working under full steam, without its train attached, can acquire a speed of 7.5 miles per hour. In order to simplify the following calculations, we will assume that the full amount of slack is taken up by the engine before the train begins to move, and the momentum

thus acquired in the locomotive is transferred to the train instantly, instead of gradually, as is actually the case. The error introduced by this assumption is inappreciable, and is on the right side. The velocity with which the train moves at the instant of starting is given by the momemtum equation $130 \times 7.5 = 2018 \times V$, from which $V = .48$. The level tangent resistance at this speed is 13 lbs. per ton; the engine tractive power at this speed is 15.4 lbs. per ton; from which we deduct 2 lbs. for acceleration, or 13.4 lbs. per ton. The force, therefore, available for overcoming grade resistances is 0.4 lbs. per ton, equivalent to a 0.02 per cent. grade.

The distance in which the train on this grade will acquire a speed of 10 miles per hour is determined from the formula deduced for momentum grades, $d = \frac{(V_1 - V_2)\, 2000,}{f - 20\ r}$ and should be calculated for increments of increase in velocity of one mile per acre.

The following table has been compiled in this manner for lines with ruling gradients from 0.5 per cent. to 1.5 per cent., giving in each case the actual grade to be adopted at the stop, the length of such grade, and the time required to acquire the minimum allowable speed of 7 or 10 miles per hour. For lines with other ruling gradients than those given in the table, the same information may be obtained with sufficient accuracy by interpolation.

TABLE OF COMPENSATION FOR GRADES AT STATIONS.

| Ruling Grade of Line.. | 0.50% | | 0.75% | | 1.00% | | 1.25% | | 1.50% | |
|---|---|---|---|---|---|---|---|---|---|---|
| Actual allowable Grade at Stop. ..... | 0.02% | | 0.31% | | 0.60% | | 0.88% | | 1.15% | |
| Amount of Reduction.. | 0.48% | | 0.44% | | 0.40% | | 0.37% | | 0.35% | |
| Minimum allowable Speed..... | Dist Feet | Time Secs | Dist Feet | Time Secs | Dist Feet | Time Secs | Dist Feet | Time Secs | Dist Feet | Time Secs |
| 7 m. per hour..... | 353 | 67 | 35 | 71 | 392 | 75 | 407 | 77 | 417 | 79 |
| 10 m. per hour..... | 763 | 100 | 83 | 108 | 909 | 116 | 991 | 124 | 1064 | 130 |

The length of the grade given in the table is the clear distance which the train must have after starting in order to acquire the necessary speed. At important stations, at which the train must stop after leaving the siding to permit of closing the switch, the grade must be lengthened accordingly.

## VIRTUAL GRADES.

The intelligent and consistent use of momentum grades may prove a great source of economy, either during construction or in improving the grades of old lines. Few railways have been built, even in recent years, in which the quantities might not have been greatly reduced by the adoption of a virtual profile in local-

ities where its use would not have impaired the efficiency of the line. The strict adherence to the maximum gradient, particularly on lines with short undulating grades, which could be surmounted by the assistance of momentum on a grade much greater than the ruling grade, has added greatly to the cost of such lines without a corresponding reduction in the cost of operation.

On lines with long ruling gradients, the saving thus effected may be inappreciable, but it is seldom that such great reductions in the grades of a line are contemplated that from thirty to fifty per cent. cannot be operated as momentum grades, whereas, if the ruling grade were strictly adhered to, much heavier work would be necessitated as well as the reduction of many grades from which no advantage is obtained in operation.

In adopting a momentum grade, great care should be taken that it be located only at a point where every assurance is to be had that the train will have acquired the speed necessary to surmount the grade. The method generally followed, of using a grade such that the total rise is equal to that of the same length of ruling gradient, plus the difference between the velocity heads of the train at its initial and final speeds, is greatly in error, assuming, as it does, that the locomotive power over and above that required to overcome the frictional and grade resistances in constant for all speeds. A grade located upon this assumption might be such as to materially limit the engine rating of a whole section, and thus defeat the purpose it was intended to serve.

A momentum grade which could be barely surmounted by one engine with a full load between certain initial and final velocities might, within minor limits, be too long for an engine of another class with a different tractive power curve. For this reason, that grade should be adopted which can be surmounted by the most unfavorable class of engine likely to be run on the section. Generally speaking, an engine in which the ratio of the tractive power at low speeds to that at high speeds is greatest, is the most unfavourable in this respect, and should therefore form the basis of calculation.

The engine herein used is a 130 ton Baldwin, compound, consolidation, with a cylinder tractive power of 29,000 lbs. at 7 miles per hour.

The distance which a train will run on a grade within certain narrow limits of velocity, the engine working under full steam, is given by the following equation of work:

$$d\ F = (V_1 - V_2)\ 2000$$

$$\text{OR} \quad d = \frac{(V_1 - V^2)\ 2000}{F} \quad - - - - (1).$$

in which d = distance in feet through which train moves between the limits of velocity.

F = average force available for acceleration or retardation in lbs. per ton.

$V^1$ = Velocity head at initial speed.

$V_2$ = Velocity head at final speed.

The cylinder power is exerted to overcome frictional and other resistances and the grade resistance. The force F is, therefore, the algebraic difference between the tractive power per ton and the total resistance per ton.

$$F = (t - r) - 20 R$$
$$= f - 20 R \qquad (2)$$

in which t = tractive power per ton at any given velocity,

r = resistance, other than grade, per ton, at any given velocity.

R = rate per cent. of grade

f = (t — r)

From this equation, Plate No. 4 was plotted, which gives the net force available for acceleration or retardation on grades from + 1.25 per cent. to — 1.25 per cent., at speeds from 0 to 40 miles per hour. The maximum speed which a train loaded for a 0.5 per cent. grade can acquire on any other grade is given in the intersection of the curve representing that grade with the zero line of tractive power.

Substituting the value of F in Equation 1, we obtain

$$d = \frac{(V_1 - V_2)\ 2000}{f - 20\ r}$$

from which Plate No. 5 was plotted.

For short increments of 2.5 miles per hour increase or decrease in velocity, we may, without sensible error, take the average of F at the beginning and end of the increment.

Similar curves have been plotted for trains loaded for 1.0 per cent. and 1.5 per cent. grades. For other grades, sufficiently accurate results may be obtained by interpolation.

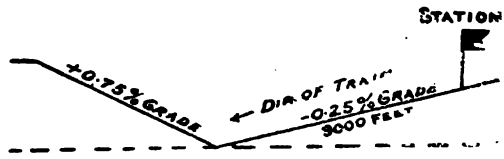



FIG. 2.

The following example will demonstrate the use of the diagrams: It is advisable, on account of a heavy increase in traffic, to reduce all the grades on a certain section of line to 0.5 per cent. There are at present several 0.75 per cent. grades which limit the engine rating, and which must therefore be reduced. One of these is situated as shown in Fig. 2, and we desire to know what length of the 0.75 per cent. grade can be surmounted by momentum before the speed of the train is reduced to seven miles per hour.

From the Acceleration Curves, Plate 5, we find that 9,000 feet on the—0.25 per cent. curve corresponds to 28 miles per hour,—the speed acquired at the foot of the grade. From the Retardation Curves, we find that 28 miles per hour on the + 0.75 per cent. curve corresponds to 2,650 feet, and 7 miles per hour to 7,300 feet (7,300—2.650) = 4,650 ft. is the distance which can be surmounted before the speed is reduced to 7 miles per hour, and beyond this point, therefore, the grade must be reduced.

To find the time required to traverse the distance, from the Acceleration Curves we find that 9,000 feet on the 0.25 per cent. curve corresponds to 5.7 minutes. From the Retardation Curves, 2,650 ft. on the + 0.75 per cent. curve corresponds to 0.83 minutes and 7,300 to 4.40 minutes. The total time from the station to the point at which the grade must be reduced is therefore 5.7 — (4.40—0.83) = 9.27 minutes.

When a grade is so long that it cannot be surmounted by momentum, but not of such length that a new line must be built. it will generally be found more economical to reduce the grade both at the foot and the summit, thus avoiding unduly heavy work at either point. The length to be reduced at the foot or the summit to give the most economical results, depends entirely upon the topography of the ground, and can only be determined by a series of trials.

The time lost in operating a momentum grade is seldom so great as to be worthy of consideration in freight transportation, but the exact amount may be directly ascertained from the diagrams, and its yearly value estimated.

## EFFECT OF GRADE IMPROVEMENTS ON OPERATING EXPENSES.

Improvements made in the physical features of a line, unless of a very extensive character, affect but little the expense of conducting passenger transportation. To within certain limits, the size of passenger trains is seldom governed by the grades, and time lost in ascending is regained in descending. The tonnage of freight trains on the other hand, is wholly limited by the ruling grades,

and any improvements which may be made in the way of eliminating curvature and rise and fall, or grade reductions, will decrease at once the cost of handling freight traffic.

To estimate, therefore, with any degree of accuracy, the yearly saving which may be realized by such improvements, we must first determine the average cost per freight train mile on that section of the road. The expenses of freight and passenger transportation are not usually subdivided in the accounts of the Company and cannot be readily ascertained without adopting a more or less arbitrary basis of subdivision. The cost of a passenger and freight train mile are not usually widely different, and for rough preliminary calculations we may ordinarily use for either unit the average cost per train mile on that portion of the line under consideration. On many lines, however, the variation is considerable, and in order to accurately estimate the yearly value of improvements in grades or curves, we must first subdivide the freight and passenger expenses, covering a sufficiently long period of time, and determine the average freight train mile cost.

A few items in the cost of transportation are directly chargeable to freight and passenger; the basis of subdivision of others is indicated by the character of the expenditure. Many items are so intimately associated with both classes of transportation that they can only be apportioned on some arbitrary basis, and it remains to adopt in the case of each such item the fairest and safest basis of distribution. For this reason it is evidently impossible to so divide the different items of railroad expense as to obtain absolute results. We may, however, obtain final units of freight and passenger train mile cost, which, though not in themselves absolute, are within a permissible limit of error, and which, for our purposes, or as a basis for comparing one division of a road against another, or a division against itself, are quite sufficiently accurate.

The several items of railroad expense are given in detail in the following classification, and, although different roads have different systems of accounting, the items are in the main similar.

GENERAL:

(1) Expenses of General Offices.
(2) Salaries and Expenses of Officers.
(3) Salaries of Clerical Force.
(4) General Expenses, Rentals, Stationery and Printing.

PRODUCTION OF MOTIVE POWER:

(5) Wages of Engineers, Firemen and Roundhouse Men.
(6) Oil, Waste and Water.
(7) Expenses of Shops and Machinery.
(8) Fuel for Locomotives.

MAINTENANCE OF EQUIPMENT:

(9) Repairs of Locomotives.
(10) Repairs of Passenger, Sleeping and Parlor Cars.
(11) Repairs of Freight and Other Cars.

LOSS AND DAMAGE:

(12) Wrecking.
(13) Injuries to Persons and Stock.
(14) Loss and Damage to Freight.

STATION SERVICE AND SUPPLIES:

(15) Agents, Clerks and Labor.
(16) Despatchers and Telegraph Operators.
(17) Station Expenses and Supplies, Heating, Lighting, Etc.
(18) Stock Yards and Elevators.
(19) Yard Rentals and Terminal Charges.

TRAIN SERVICE AND SUPPLIES:

(20) Conductors, Baggagemen and Brakemen.
(21) Care, Heating and Lubricating Cars.
(22) Train Supplies.
(23) Wages and Supplies, Parlor and Sleeping Cars.

MAINTENANCE OF WAY:

(24) Maintenance of Buildings, Docks, Wharves, Telegraphs, Fences, Etc.
(25) " Stockyards and Elevators.
(26) " Bridges and Culverts.
(27) " Road Bed.
(28) Repairs of Track, Switches and Frogs.
(29) Renewals of Ties.
(30) Renewals of Rails and Fastenings.
(31) Incidental Expenses.

We will now proceed to apportion the several items above between freight and passenger, according to the following measures, when they cannot be directly charged to one or the other account:

In the proportion of the Freight and Passenger Revenue.

In the proportion of the Freight and Passenger Train Mileage.

In the proportion of the Freight and Passenger Engine Mileage.

In the case of the train and engine mileage of mixed trains, which is at most but a small proportion of the total on all roads, when more accurate data is not obtainable, they may be divided in the proportion of two-thirds to freight and one-third to passenger. Switching engine mileage is divided in the proportion of eighty per cent. to freight and twenty per cent. to passenger.

Items (11), (14), (18) and (25) are chargeable directly to freight.

Items (10) and (23) are chargeable directly to passenger.

Items (1) (2), (3), (4), (15), (17), (19) and (24) are divided in the proportion of the freight and passenger revenue.

Items (12), (13), (16), (26), (27), (28), (29), (30) and (31) in the proportion of the freight and passenger revenue.

Item (9) in the proportion of the freight and passenger engine mileage.

Items (20), (21) and (22) are directly proportional to the train mileage, but the cost of these items per train mile varies from 50 to 70 per cent. more in freight than in passenger service. In the absence of more accurate data, therefore, we add 60 per cent. to the freight train mileage, and distribute the items in the proportion of the freight train mileage thus obtained, to the actual passenger train mileage.

Items (5), (6) and (7) vary directly with the engine mileage, but their cost per freight engine mile is from 60 to 100 per cent. greater than per passenger engine mile. To apportion these items, therefore, when more accurate data is not obtainable, we increase the freight engine mileage 80 per cent., and adopt this and the passenger engine mileage as the basis of subdivision.

The item "Fuel for Locomotives" (8) is one of the largest of railroad expense, and should be very carefully distributed. If access is to be had to the Company's accounts, the relative amounts chargeable to freight and passenger may usually be readily ascertained. Generally, this item may be apportioned arbitrarily with considerable accuracy. The cost of fuel per engine mile varies from 50 to 100 per cent. greater in freight than in passenger service. To the freight engine mileage, however, has been added the switching mileage, which reduces the increase to an average of about 50 per cent. The item is therefore divided in the proportion of the freight engine mileage, increased 50 per cent., to the actual passenger engine mileage. Having now apportioned the total expenses between freight and passenger, these divided respectively by the freight and passenger train mileage give the average cost per freight and passenger train mile. We may proceed further and divide these expenses respectively by the total number of revenue ton and passenger miles, and obtain the average cost per ton and per passenger mile.—the main transportation units.

The cost of freight and passenger train mileage is found to vary from 75 cts. to $1.50 and from 65 cts. to $1.40 respectively, according to conditions, but a fair average for each may be taken as 85 cts.

and 75 cts. Having obtained these results, we are in a position to estimate the yearly value of improvements by which the engine rating is increased. Proceeding upon the lines laid down by Mr. A. M. Wellington, it will be found that from 50 to 60 per cent. of the expenses varies with increased train mileage. This percentage, multiplied by the cost per freight train mile and by the number of train miles saved in a year, gives the yearly saving in operating expenses.

When the tonnage of a train is not governed by the speed or other limiting conditions, the haulage capacity of a locomotive on different grades may be determined from the following formula:

$$T = \frac{t}{4.5 + 20\,r} \; E$$

in which

T = tonnage of train.
c = cylinder tractive power of engine at a minimum allowable speed of 7 or 10 miles per hour.
r = rate of grade.
E = weight of engine in tons.

The engine rating may be greater in one direction than the other, and at the same time the traffic may be of such a nature, that the average train tonnage may be less in the former direction than in the latter; consequently, although the rating may be considerably increased in one direction, it does not necessarily follow that the average train tonnage in that direction will be increased in a like proportion. In estimating, therefore, the number of train miles saved by an increase in the engine rating, care should be taken to ascertain the actual number of properly balanced trains necessary to operate on the new line to handle the given traffic.

The question of the most suitable grades to adopt is usually complicated by an unequal balancing of the grades to the traffic. If the tonnage is greater in one direction the return tonnage will consist partially or wholly of unloaded cars. It is usually estimated that 30 per cent. additional power is required to haul empty cars than a like tonnage of loaded cars, and the locomotive haulage sheets are compiled on that basis. The frequency with which trains of empty cars are stalled would indicate, however, that this is not a sufficiently large allowance. Wellington's experiments on loaded and empty cars tend to confirm this belief, giving 45 per cent. at 7 miles per hour as the additional power required, a percentage which decreases slightly as the speed increases. In reducing grades to balance the traffic, therefore, that portion of the return tonnage which consists of empty cars should be increased by at least 35 or 40 per cent., and proportionately for partially loaded cars, and grades adopted which

will suit this equivalent tonnage and the actual tonnage in the opposite direction.

On lines traversing a river valley with a low ruling gradient of a practically continuous rise, the scheduled speed may be such as to greatly reduce the train tonnage below that which could be hauled at low speeds on the ruling grade. In such cases, the horse power from the engine and the speed determine the tonnage of the train which may be calculated from the following formula:

$$T = \frac{375\ P}{S \times R} - E$$

in which

T = tonnage of train.
P = horse-power of locomotive at average speed.
S = average speed in miles per hour.
R = average resistance of train in lbs. per ton.
E = weight of engine in tons.

## REDUCTION OF RISE AND FALL.

The cost of different classes of rise and fall depends upon the rate of grade which is surmounted and the ruling grade of the section. If the ruling grade is equal to, or less than the grade of repose, which is taken as 0.5 per cent., it is not considered that rise and fall adds appreciably to the ordinary cost of operation, since such grades (unless of great length) can be operated under uniform steam, the speed alone varying. The cost of wear and tear is but slightly affected, and the cost of fuel is practically uniform; the potential energy stored in ascending a grade is given out in descending, and except as wasted by stops the kinetic energy of the train, due to the speed acquired in descending a grade, assists in ascending the next. On such grades, the cost per foot of rise and fall is estimated at 24 cents per daily train, per year.

On grades from 0.5 to 0.8 per cent. which do not require the use of sand in ascending and which in descending are neither operated under steam nor require the use of brakes, a foot of rise and fall is estimated to cost 71 cts., if on a minor grade, and $1.42 if on a ruling grade.

On grades greater than 0.8 per cent. which are of too great length to be operated as momentum grades and which require the use of both brakes and sand, a foot of rise and fall on a minor grade is estimated at $2.27, and if on a ruling grade $2.98.

The above figures are on the basis of an average train mile cost of 85 cts., and should be increased proportionately to the actual train mile cost on any particular section. and represent the yearly cost of one foot of rise and fall per daily train each way. (If the rise and

fall on the new line is less than that of the old line, the number of trains used in the calculation will be that operated on the old line, and vice versa).

In estimating the amount of rise and fall which may be avoided by a change of line, care should be taken to allot each to its proper class, which does not necessarily depend upon the actual rate of the grade. A grade of short length, but of a rate much greater than the ruling grade, may be in such position as to be operated by momentum, and the value of its rise and fall will, therefore, be reduced to that of a minor gradient.

## REDUCTION OF CURVATURE.

The most serious objection to curves is the sentimental prejudice of the public against them. To a large extent the distinctive feature which has enabled the rapid growth and development of American railways has been the recognition of the fact that curvature, consistently introduced, was not a disadvantage. When of ordinary sharpness, properly elevated, lined and surfaced, curves may be traversed with as great a degree of comfort and safety as tangents, and if provided with easement ends, the transition from the tangent to the curve is imperceptible.

The actual cost of operation is but slightly increased by additional curvature up to the limit when sharp curves necessitate a reduction in the speed of fast trains due to the slightly increased liability to derailment. Excessive curvature should, however, be avoided wherever possible, and the necessity for using a maximum curve at one or more points in an engine stage is not, as in the case of maximum grades, sufficient justification for its general adoption.

At the most, the money value of eliminating curvature is of but minor importance from the standpoint of economical operation only. Wellington estimates that 36 per cent. of the working expenses vary with the curve resistances, which average about one-half pound per ton per degree. Taking the average level tangent resistance as 6 lbs. per ton, this is equivalent to a continuous 12 degree curve, or 633 degrees of curvature in a mile. Assuming the cost of a train mile as 85 cts., the yearly value per daily train each way for eliminating one degree of curvature is:

$$\frac{85c \times 36\%}{633} \times 365 \times 2 = 35.3 \text{ cts.}$$

(The number of daily trains used in this estimate will be that operated on the old line, or necessary to handle the same volume of traffic on the new line, according as the curvature is greater or less on the old than on the new line).

A reduction of curvature, unlike a reduction of grades, always involves the construction of more or less new line, so that from the smallness of the above figure it is evident that the elimination of curvature alone, unless attended by a reduction in the ruling grade or a shortening of distance, is seldom, if ever, warranted by the consequent saving in operating expenses.

Having now estimated the yearly value in operation of each of the several kinds of the proposed improvements, the amounts in favour of or against the alterations may be summarized for comparison as follows:

ESTIMATE OF ECONOMIC VALUE OF IMPROVEMENTS.

From..... To.....

Description of work.....

| NATURE OF CHANGE. | Present Line | Proposed Line | Difference | Yearly Value |
|---|---|---|---|---|
| DISTANCE | | | | |
| " Effect on Operating Expenses | | | | |
| " " Passenger Receipts. | | | | |
| " " Freight " | | | | |
| Actual Grade....compensated....bound | | | | |
| " " .... " ....bound | | | | |
| Virtual " " ....bound | | | | |
| " " " ....bound | | | | |
| Rating of....class Locomotive ...bound Tons | | | | |
| " .... " ...bound " | | | | |
| Rise and Fall, on maximum grade Fee | | | | |
| " " medium " "t | | | | |
| " " minor " " | | | | |
| CURVATURE DEGREES | | | | |
| Time Saved by Pass'r Trains....bound Min. | | | | |
| " " " ....bound " | | | | |
| " Freight " ....bound " | | | | |
| " " ....bound " | | | | |
| Improvements in Alignment. | | | | |
| Total Yearly Saving in Favour of Improvements | | | | |

When radical changes of this character are contemplated, it is usual at the same time to improve the standard of the track. The

cost of such work is not, however, properly chargeable to the improvements, and for the purposes of comparison should be deducted from the total estimated cost of the proposed changes. The balance, chargeable to improvements, divided into the estimated yearly saving, gives the rate of interest on the expenditure.

In this manner a number of suggestions for improving a line may be reduced to their money equivalents and that proposition adopted which, consistent with other requirements, will pay the largest percentage of interest on the investment.

## DISCUSSION.

Mr. D. Macpherson remarked that the author had certainly compressed a good deal of matter within a small space, and had outlined a seemingly very practical method of arriving at the economic value of railway improvements. If all his hypotheses were accurate the paper should be a valuable contribution to our proceedings. It did not seem, however, that the attempt to reduce to actual dollars and cents the exact value of all the elements which enter into the improvements of a piece of line could serve as more than a rough guide as to how such work should be approached. It could scarcely be accepted as correct in specific cases without very careful scrutiny of all the conditions which would modify the different units of value. The definition given of Civil Engineering:—"The art of making a dollar earn the most interest," did not appear a very happy one, as, although it might fit specific works, it might equally well apply to the unlearned profession of money lending at high rates of interest. No one would dispute the author's statement that, generally speaking, the most economical line to adopt was the one in which the sum of operating expenses and interest on whole outlay was least, provided the business available was as great or in proportion to what would be available for a more expensive line. The assertion that engineers were prone to work to extreme limits of grades and curves, where easier grades and curves could easily be obtained, was, in his opinion, incorrect and uncalled for. It was, no doubt, the fact that some poor engineers did so; but, in the majority of cases where it might seem to be the case, it would be found that the financial men in authority insisted upon the cheapest initial line, regardless of future consequences. With regard to the statement that the shortening of a line would reduce the revenue from non-competitive local traffic, this was founded on the idea that such traffic was usually handled on a mileage basis, whereas the fact was that it was oftener handled on arbitrary ratings, which to a large extent ignored distances. Thus, shortening the line would reduce operating expenses and increase net revenue. The author's reasoning in connection with the using of virtual grades was a clear statement of a most important feature in railway operations. In times gone by, no doubt, many engineers paid little attention to this most scientific part of location, but these days of keen competition made it absolutely necessary to get the maximum haulage capacity out of our engines at the least possible expense. This could only be done with the judicious use of

virtual grades. In his opinion the author had, on the whole, made a very creditable and interesting demonstration of a very complex subject.

MR. COLPITTS, in reply to Mr. Macpherson, remarked that it was obviously impossible, in a general discussion of the subject, to specify precise units that would be applicable in all cases, for there was not only a great variation in such units on different roads, but on different portions of the same road. In no two instances, perhaps, did similar conditions exist, and each case must be treated according to circumstances. Certain factors, quite outside the features herein discussed, might have a large bearing on the question, and these must be weighed as the peculiar conditions warranted; whereas to treat such matters in a general way would be valueless. The paper was intended to define the methods by which the units of the various items, which were common to all roads, might be determined in specific cases, and, when determined, the manner in which they should be applied. The results obtained in such an investigation should assuredly be considered in conjunction with all other conditions, local or otherwise, which might have weight; and, while it was not to be expected that the investigation could be made with absolute accuracy, it might be sufficiently accurate to determine within very narrow limits a course of action, which was what was sought. The effect on operating expenses of a minor change in distance was discussed under that head and its value estimated independently. The receipts from non-competitive local traffic, that was handled on an arbitrary basis without regard to the actual distance hauled, would not be affected by slight changes in distances. On traffic of this nature, however, on which the rates were established on a mileage basis, the receipts would be reduced or increased in direct proportion to the amount by which the line was shortened or lengthened, and it was this traffic which was considered under Class I.

Thursday, 7th November.

MR. G. H. DUGGAN, Vice-President, in the Chair.

**Paper No. 161.**

## THE CRITICAL VELOCITIES OF WATER.

BY DR. E. G. COKER, A.M. CAN. SOC. C.E.

It is well known that when water issues freely from a circular orifice, whose diameter is small compared with the head of water upon it, that every part of the jet is moving with the same, or nearly the same velocity, viz., that due to the head of water in the tank or vessel less a small quantity due to the friction of the orifice and the resistance of the air, which in general is negligible.

Each particle of water has the same force impressed upon it, and if we could trace the path of any small element we should expect to find that it moved parallel to the axis of the jet, or, as it is called, in stream line motion, except close to the opening where the jet contracts owing to the meeting of water coming from various parts of the interior of the tank or vessel.

Although this seems simple enough it is desirable to confirm this view by direct experiment, and we are at once met by the difficulty that, although we can see through the jet, we cannot see into it, so that, if the motion is not stream line motion, we cannot detect it. If, however, we colour a portion of the water we can follow the motion, and this can easily be done by introducing a thin stream of colour into the jet by a glass tube placed at the back of the orifice. If care be taken to give the coloured stream the same velocity as the issuing jet the colour will appear as a clearly defined band in the jet, proving that the motion is stream line, and, whatever be the velocity, the band will always be distinct if the proportion of colour be not too great.

If, however, the same jet were caused to move through a pipe we have a possible disturbing cause in the enveloping surface, and this may cause the motion to be quite different. We can, as before, examine what happens by using a glass tube and a colour band to indicate the motion.

*Abstract of a Lecture on "Critical Velocity of Water."*

If we start with a small velocity in the tube the colour band will be clearly defined as before, and will continue to remain so until a certain velocity is reached, when suddenly, without any warning, the colour band disappears and we see a mass of eddies in the tube.

This phenomenon was discovered by Professor Osborne Reynolds, and he called the velocity at which the motion changed from stream line to eddy or turbulent motion "critical velocity."

Clearly the cause is to be looked for in the walls of the pipe. and it has been shown by several experimenters that water enclosed in this way does not move with the same velocity throughout the section, but has the greatest velocity at the centre diminishing to nothing at the walls, the curve of velocity being somewhat as shown in Fig. 1.

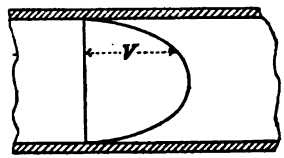



Fig. 1

We have, then, water moving with different velocities at different parts of the pipe, and we, consequently, get sliding between the different layers, and there is a possible cause of disturbance which increases as the velocity increases. On the other hand, water is not an ideal liquid, as it possesses in a small degree the property (which is so marked in glycerine, treacle and oils) of viscosity, and this, in some way, resists the tendency to upset the motion, so that we get stream lines below a certain velocity and turbulent or eddy motion above. Not only does the motion change, but the law of resistance changes also, from resistance varying as the velocity with stream line motion to resistance varying nearly as the square of the velocity with eddy motion.

The factors which determine the critical velocity have been carefully examined by Osborne Reynolds, and he has shown that the determining causes are temperature and size of pipe, or, expressed in the form of an equation,

$$V_c = k \frac{f(T)}{D}$$

Where $V_c$ is the critical velocity, D is the diameter of the pipe, and

$$f(T) = (1 + \cdot 03368\ T + \cdot 000221\ T^2)^{-1}$$

T being the temperature centigrade, and k is a constant which, if feet and seconds are used, has the value 0.245.

METHODS OF DETERMINING CRITICAL VELOCITY.

The sudden break down of the colour band in a glass pipe affords a means of determining the critical velocity in glass pipes but is not applicable to metal ones; but, since the law of resistance changes suddenly at the critical velocity, it is not difficult to determine the change point and from it the critical velocity.

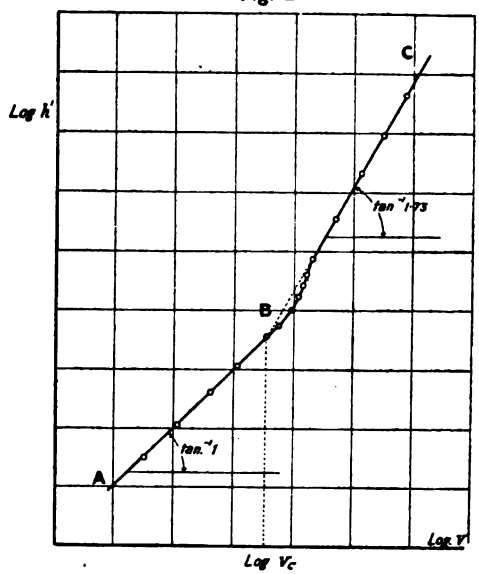
The loss of head $h^1$ in a length of pipe conveying water at velocity v can always be expressed in the form $h = c V^n$ where c is a constant depending upon the dimensions of the pipe and the material, and $n$ is the power of the velocity, which has one value below the critical velocity and another above. If, therefore, corresponding values of $h^1$ and v be determined on each side of the critical point, the point can be determined by plotting.

The labour is much reduced if logarithmic co-ordinates are used, for since

$$\log h^1 = \log c + n \log V.$$

the value of $n$ is now expressed as the tangent of the inclination of some straight line, and the critical point is fixed by the intersection of two such lines.

Fig. 2



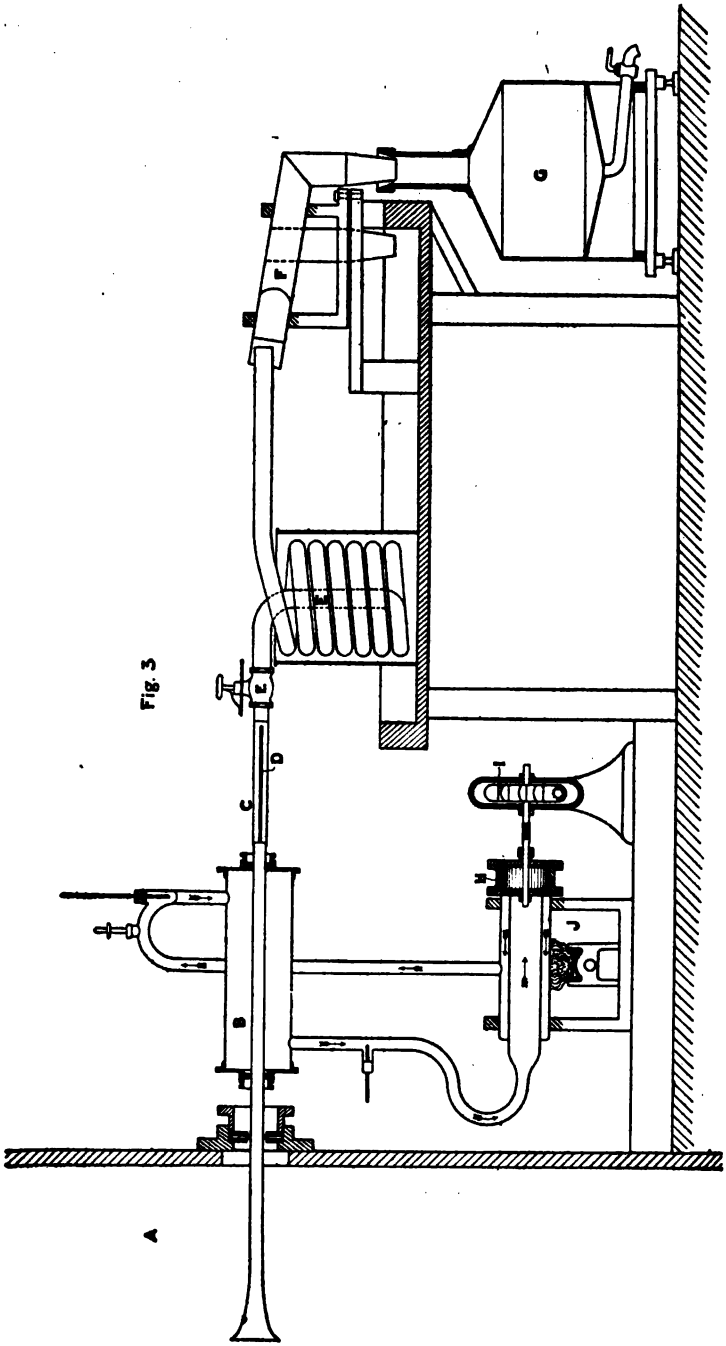



Fig. 3

A typical diagram illustrating the method is shown by Fig. 2, in which the inclination of line A.B. is forty-five degrees to the axis, and therefore $n = 1$ and BC has an inclination $\tan^{-1} n$ where $n = 1.73$, the abscissae corresponding to B determines the value log. $V_c$, and hence $V_c$

A third method, which has been developed by Dr. Barnes, and the author takes advantage of the phenomenon first noticed by Dr. Barnes, that if water is heated while flowing through tubes in stream line motion, the distribution of heat throughout the water column is not uniform. If heat be applied to the outside of a tube only the layer in direct contact with the tube will be heated, the great bulk of the water remaining unheated; but, when the critical velocity is reached, the break down of the stream line motion causes a complete mixing of the water in the tube and, consequently, a rise of temperature, which is easily detected by a mercurial thermometer. Experiment showed that the rise of temperature coincided with the break down of an introduced colour band.

In the actual experiments the apparatus took the form shown in Fig. 3, in which A is a large tank into which projected the tube under experiment, having a metal trumpet flare to ensure undisturbed flow.

The protruding end of the pipe was threaded through a water jacket, B, with ends closed by stuffing boxes, and the outer end of the pipe was fitted with a glass prolongation, C, containing a mercurial thermometer, D. The water was afterwards led through a regulating valve, E, and a cooling coil, E, to a two-way switch over F having one pipe leading to waste and the other to a double cone measure, G, whose capacity was known very accurately.

Water was circulated continuously through the jacket by a centrifugal pump, H, operated by a Pelton water motor, I. A gas flame, J, under the pump, served to regulate the temperature of the jacket.

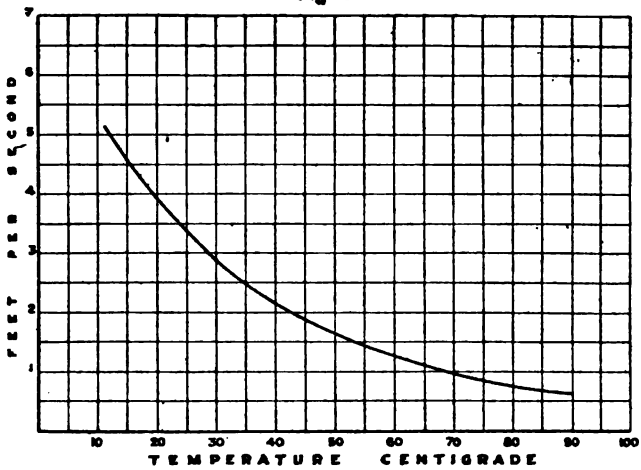
In making an experiment the water in the jacket is brought to a temperature slightly above or below the water flowing through the pipe. As long as the velocity of flow remains below the critical velocity no change can be detected, but as soon as the critical velocity is reached the thread of mercury in the thermometer jumps to a new position and remains there as long as the unsteady motion continues.

The velocity at which the change occurs can be readily determined by causing the discharge to flow into the measuring vessel for a known interval of time, and from the dimensions of the pipe the critical velocity can at once be determined.

As an example of the results obtained by this method Fig. 4 shows the variation with temperature of the critical velocity in a brass pipe having a diameter of 0.0345 feet. The range of temper-

ature in this case extended over more than seventy degrees Centigrade, and the critical velocity varied from little more than 0.5 feet per second to 5 feet per second.

Fig. 4



Thursday, 21st November.

MR. H. IRWIN, Member of Council, in the Chair.

Messrs. J. W. Heckman, E. S. Mattice, G. H. Garden, H. Irwin and C. H. McLeod, having been appointed scrutineers of the ballot for the election of Members, reported the following elected:—

MEMBERS.

JAMES T. CHILD. A. C. DENNIS.
J. S. DENNIS.

ASSOCIATE MEMBERS.

H. A. BAYFIELD. LESTER W. GILL.
H. L. JORDAN. R. H. PARKINSON.
H. A. ROWE. A. W. H. STIMPSON.
W. V. TAYLOR. J. H. WALLACE.

ASSOCIATE.

E. McG. QUIRK.

Transferred from the class of Associate Member to the class of Member:—

A. H. N. BRUCE. M. W. HOPKINS.
A. J. GRANT. J. W. LE B. ROSS.
C. C. WORSFOLD.

Transferred from the class of Associate to the class of Associate Member:—

HENRY F. DUCK.

Transferred from the class of Student to the class of Associate Member:—

C. W. ARCHIBALD. A. J. MCPHERSON.

STUDENTS.

| | |
|---|---|
| K. McK. CAMERON. | D. J. CARTER. |
| A. R. CHIPMAN. | R. H. HARCOURT. |
| G. B. HUGHES. | T. C. KEEFER, JR. |
| C. C. McLENNAN. | F. H. MAYNARD. |
| J. MAX MUNDIE. | H. F. OSLER. |
| J. M. OXLEY. | W. F. SPARROW. |

A. STEWART.

**Paper No. 162.**

## SUPERSTRUCTURE OF THE INTERPROVINCIAL BRIDGE, OTTAWA.

By F. P. Shearwood, A.M.,Can.Soc.C.E.

The bridge recently erected at Nepean Point, Ottawa, connecting as it does two important commercial towns, as well as two provinces, naturally attracts the attention of the ordinary observer and is especially interesting to the engineer.

The bridge and approaches consist of the following structures shown in Plate 8.

| | | |
|---|---|---|
| 1. The Southern approach including— | | |
| Trestle at St. Patrick Street.. .. .. .. .. .. .. | 360' | |
| Skew plate girder span.. .. .. .. .. .. .. .. .. | 30' | |
| | | 390' |
| 2. The main crossing of the Ottawa River including— | | |
| Two plate girder spans.. .. .. .. .. .. .. .. | 59' | |
| Cantilever structure.. .. .. .. .. .. .. .. .. .. | 1,049'-9" | |
| 1 Truss span .. .. .. .. .. .. .. .. .. .. .. .. | 247' | |
| 1 Truss span.. .. .. .. .. .. .. .. .. .. .. .. | 140' | |
| | | 1,495'-9" |
| 3. The Northern approach including— | | |
| Trestle to first roadway .. .. .. .. .. .. .. .. | 390' | |
| Plate girder span over first roadway.. .. .. .. | 60' | |
| Trestle from first roadway to Laurier Ave .. | 274' | |
| Skew plate girder over Laurier Ave.. .. .. .. | 75' | |
| | | 799' |
| Total .. .. .. .. .. .. .. .. .. .. .. .. .. .. | | 2,684'-9" |

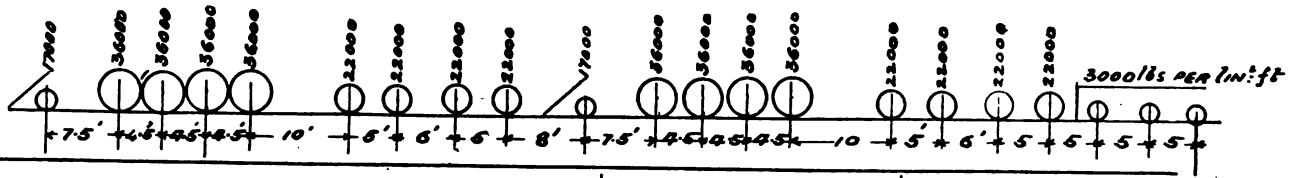
The Interprovincial Bridge Company stipulated that provision should be made for a single track steam railway, two lines of electric railway, two roadways for vehicles and two footpaths. The arrangement of these as finally agreed upon is shown on plate 12.

The approaches on the Southern end are designed to carry three steam railway tracks. The electric track on the east side of the bridge when it reaches the Southern abutment ascends by a surface track to the summit of Nepean Point Park. The Western track, descending to the level of St. Patrick street, crosses underneath the trestle which carries the steam railway tracks.

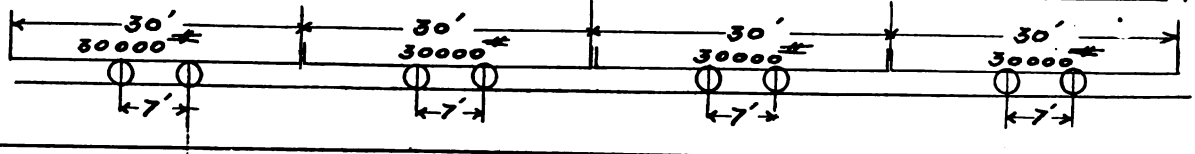
The electric track roadway and footpaths on reaching the Northern abutment descend by a 5 per cent. grade to the level of the ground at the first roadway crossing. The trestle beyond this point supports only a single track railway.

The design for the superstructure was made by the Dominion Bridge Co., Ltd., in accordance with the following loads adopted by the Interprovincial Bridge Company and approved by the Dominion Government.

RAILWAY LOAD.

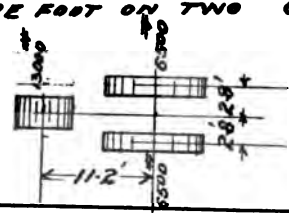



ELECTRIC Ry. LOADS




ROADWAYS = 40 POUNDS PER SQUARE FOOT ON TWO 8 FOOT ROADWAYS, FOR STRESSES IN TRUSSES

OR ROAD ROLLER OF




FOR STRESSES IN FLOOR SYSTEM

FOOT WALKS = 20 POUNDS PER SQUARE FOOT ON TWO 5 FOOT SIDE WALKS, FOR STRESSES IN TRUSSES.
OR 70 POUNDS " " " " " " " " " IN FLOOR SYSTEM.

In other respects the Dominion Government specifications of 1896 were followed.

The trusses, for the sake of stability are spaced 24 feet apart from centre to centre, the steam railway being placed in the centre and the foot walks between this and the trusses in order to utilize what would otherwise be vacant space.

The general arrangement of the suspended span trusses was to a large extent determined by the Dominion Government stipulation, that there should be a clear waterway above H. W. L. of 45 feet, and by the desire of the Interprovincial Bridge Company, that the base of rail should be kept as low as possible, in order to avoid grades in the approaches.

In order to save masonry in the main piers the shoes were placed as near as permissible to the river surface and the lower chords inclined. This was found not to be uneconomical in the steel weight, and had also the advantage of reducing the total height of the main towers.

To obviate the difficulty encountered in the suspended span, by reason of the short distance between the bottom of the steel and the base of rails, and also on account of the shoe detail used at the northern end of the anchor arms, a greater distance was taken between the centre line of the lower chord and the base of rail of anchor arms. This necessitated a slightly unsymmetrical angle of rise in the chords on either side of the towers, but on account of the long overhang of the floor beam brackets, it was thought to be unobjectionable and has proved to be unnoticeable in the structure even before the floor was laid.

The question of the wind stresses also presented an interesting aspect of the structure. A wind pressure of 30 lbs. was specified and the surface it acted upon was represented in the usual way as twice the area of the trusses and once the floor system as seen in elevation. A moving train of 10 feet in height was also taken into consideration and placed so as to give the maximum stresses in any member.

The stresses in the top laterals from the suspended span are transferred to the top lateral trusses of the cantilever and are not conveyed in the usual method by the end post bracing of the suspended span to the bottom lateral system. The object in doing this was to distribute the wind stresses more evenly between the upper and lower chords and also to obviate the difficulty of providing an efficient shear joint for a large stress at one point.

The top lateral shears are figured to pass down the heavy inclined members S 6-7 and S 8-9. To insure this result the laterals in panels 6 and 9 are omitted. The upper laterals are designed to resist tension only, but are latticed vertically for the sake of appearance. The lower laterals are designed to act both in tension

and compression. This was found more economical and was adopted in order to simplify the difficulty of providing for secondary stresses induced at the pin joints. Sway bracing is used between all compression members where possible, that between the inclined members S 6-7 and S 8-9 being designed to resist the shear of the upper laterals connecting to them. The sway bracing in posts P 8 has only to transfer the shear from the laterals in panels 7 and 8. The wind pressure from the floor and train is transferred by knee braces connecting the floor beams to the lower chords.

The cambering of the suspended span was accomplished in the usual manner by lengthening the upper chords, but in the anchor and cantilever arms each member was corrected for its own alteration of length due to the dead load stresses and pin play.

The deflection due to live load at the ends of the cantilevers was compensated for by shortening the upper chords in panels 6 to 9. The bridge as finished shows between piers 2 and 3, a slight rise in the floor on a fairly regular curve. The anchor arms also have a slight, but noticeable camber.

Plate 10 shows in detail the construction of the anchor arms which were made in such a way, as to be self supporting before any of the cantilever arms were erected, as it was quite possible that the scows used to support the false work might be withdrawn before the erection of any part of the cantilever. This required a temporary strut to be placed in panel No. 7 (shown by the dotted line in Plate 9) and also that tension should be provided for at the lower chord splices and in the connection of S 3. The bottom connections of the suspended posts were reinforced to support the traveler during erection. The lower chords of the anchor and cantilever arms were constructed with three ribs for the purpose of reducing the thickness of the material and consequently undesirably long rivets. In calculating the stresses in the pins, they were considered as continuous girders, resting on three rigid supports formed by the ribs of the lower chords. The other members were assumed to be loaded in proportion to their areas.

The tie plates on the lower chords at the pin points were designed to redistribute the increments of stress received through the pin into the three ribs proportional to their areas, and also to transfer the increments of stress from the laterals. At points 2, 4 and 6, the lower laterals are connected to the chords with no tie plates occuring at the point of intersection on the upper side of the chord, and the lateral plates are strengthened to resist the bending induced by the transference of the shear from one lateral to the next.

Plate 12 shows in detail a portion of the cantilever arms and also a part of the suspended span, P 12 representing the hangers

to carry the suspended span and at the lower part shows the difficulty already referred to, due to the extremely short distance between the floor and the bottom of the steel. The heavy inclined members at this point as well as at No. 12 interfered with the abutting joint of the floor beam as used at the other panel points of the bridge, and the floor beam was deepened to straddle the chord and to obtain a compression splice below it. The details at the connection of the suspended span L 13 are shown with the erection wedges in place where the pin bore on two small forgings which were removed when the centre span was connected and which bore on several shims. These shims were used as being the best way of adjustment compatible with the method used for erection and in their turn bore on stiffener angles riveted to the chord L 12.

At U 14 is situated the upper adjustable joint, and in order to resist the tension during erection two adjustable yokes engaging the pin were supported by four diaphrams. Between the diaphram and the yoke nuts, shims were placed and slotted to enable them to be easily applied when starting to erect the suspended span and removed when it was connected. The top lateral connection at this point is shown in the plan view, T R 14 being riveted to the chord U 14, is held from horizontal movement by the plates which connect the top strut T S 14 to the chord U 13, being fitted to bear against the chord 14. The top strut, T S 14, then takes the shear from the top lateral system of the suspended span and transfers it to the top lateral truss of the cantilever arm. It will be noticed that the lateral connection plates on both chords are extended to meet the tie plates in order to reinforce the ribs against bending from lateral stress.

At point U 13, a somewhat intricate connection for the laterals is shown, TR 13, necessarily being the same width as the chord U 13 while TR 12 is wider on account of other top lateral connections. The connection plates of TR 13, are riveted to the chords U 13 and TS 13. TS 13 is connected directly with the succeeding lateral TR 12. At this point it was inconvenient to intersect the centre lines of the horizontally stressed members; but the small bending is easily resisted by the wide members of the vertical trusses.

At point U 11 is seen the method of utilizing the post channels to make a stiff and efficient connection for the laterals. The top strut is placed slightly away from the pin centre in order to connect to the sway bracing which is kept on the centre line of the posts.

Plate 12 illustrates the means of transferring the wind stresses from the suspended span to the cantilever arms, P 13 and FB 13 are connected to L 13 while L 12 has slotted connection to allow for the expansion and contraction of the span.

The stringers in panel 13 are riveted to the floor beam FB 13, but those in panel 12 are only supported on brackets on which they slide to accommodate the expansion and contraction. The shear from lateral R 13 is transferred to the floor beam by 12″ channels and this is held from moving by the brackets placed on each side of the stringers S 12, which are riveted to the bottom strut BS 13, thus transferring the shears directly to R 12. The angle bracing between the stringers S 13 is to relieve the floor beam from the bending induced by the horizontal load applied on the brackets. This method of transferring the wind shears enables shears to be perfectly distributed between the two sides, and the laterals being designed to resist tension and compression, only half the total stress is resisted in each case, and the compression wind stresses in the main chords are slightly reduced.

In the cross section of the floor beam is seen the construction of the floor beam brackets for the sliding support of the stringers, and the tension at the top of the brackets, is provided for by a splice plate passing through a slot in the floor beam web and riveted to the fixed stringer.

The details of the construction on pier No. 4 are shown on Plate 12. To prevent the use of a very wide pier and the eccentric placing of the anchorage, the ends of the cantilever and fixed spans were made to rest on a common shoe. Thus the dead load of the fixed span was used directly to supply part of the anchorage required for the cantilever.

It will be observed that short pieces of channels placed endwise are inserted in the centre of the upper tier of beams of the shoe, which are to resist the horizontal stresses due to the resistance made by the rollers to the expansion and contraction of the spans and so obviate bending stresses in the webs of the eye beams.

To carry the horizontal stresses to the masonry without calling on the ribs of the shoe to do the work for which they could hardly have been made capable, a series of struts was used, which is shown in the half cross section in Plate 12. The laterals of the fixed span are connected with the bottom flanges of the floor beams, the shear being transferred by the inclineal strut to the bottom strut at the ends of which the anchor arm laterals are connected. On the lower flange of this strut, to which all the lateral shears have been transferred, a steel casting was bolted, which slides longitudinally in a casting bolted on a two webbed girder, which is firmly anchored to the masonry, and by which all side movement is resisted.

Plate 11 represents the portal bracing situated between the inclined members S 6-7, which differs only from the bracing between S 8-9 in that the main member S 8-9 is slightly wider back to back of the flange angles.

The object of this bracing as already stated is to resist the shears from the top lateral system. The top lateral connection plate connects also the top strut, upper chord, and S 6 by which the components of the stress from T R 5 are transferred direct to their proper channels and not by relying on the friction of the pin.

Owing to the necessity for providing sufficient head room over the railway tracks, the diagonals resisting the wind shear do not intersect on the centre line of S 7, and consequently this member is reinforced by four angles, riveted to the webs and tie plates, in order to resist bending. As there was a large amount of shear induced between these points, it was found inadvisable to allow the webs of S 6-7 to remain unconnected for any appreciable distance, and so the working members CS 7 and P 7, which were obliged to be connected with the pin, were passed through slots in the heavy tie plates connecting the webs. The other two, which are merely stiffening members, are riveted at this point to lug angles connected with the tie plates. The extra size of the lattice in S 7 is to provide for the wind shears. At the lower end of S 7, the diagonals are connected as low as possible. Tie plates and a diaphram are introduced to reinforce the member, and the shear is transferred by a gusset plate, which connects directly with the bottom strut shown on the Plate.

Plate 11 also shows the details in connection with the posts and the shoes on the main piers. The post P 8 has its webs placed in the opposite direction to that used in the other members of the bridge.

At the joint U 8, to reduce the bending on the pin and also the thickness of the bearing points, two diaphrams are inserted between the webs, and bearing plates are riveted to the flange angles, forming in all four points of bearing for the pin. Two of the pin plates of the diaphrams are extended beyond the pin to connect with the top strut. The floor beam at this panel point passes between the webs of P 8, resting on a diaphram riveted to the webs of the post.

To accommodate the packing of the members at pin L 8. P 8 was spread so as to bear on the pin outside of the lower chords which abut on the pin opposite each other. One pair of plates on each running past the pin to resist a possible tension from the wind force, and S 8 is made slightly wider than S 7. The ribs of the cast steel shoes do not encircle the pin, as no uplift can possibly occur at this point until the wind shall blow at a pressure of 77 lbs. per square foot on the total surface, and this is assuming that the anchorages do not assist in the resistance. The carrying of the wind force to the masonry is accomplished by all the members which have lateral stresses connecting with the cross strut BS 8, as seen in the drawing P 8, S 7 and S 8, having gusset plates riveted to

them and to the upper connection plate of BS 8, the bottom laterals are riveted directly to the connection plate of BS 8, therefore, all the lateral shears are concentrated in BS 8. This is connected by gusset plates to the upper tiers of the beams under the shoes. The lower tier has two lines of separators and also is filled with concrete.

A typical floor beam at panel No. 16 is shown on Plate 12, the tension splice at the support for the cantilevered bracket is made by riveting splice plates to the flanges which pass through slots made in the post channels. The posts are reinforced at these points, the sectional area through the slot being 50 per cent. in excess of the net area required in the member.

The compression joint is made by facing the two abutting flanges which bear on the post, the two webs of which are separated by a small casting accurately fitted between them. The diaphram between the post channels is figured to transfer the greatest difference in shear between the brackets and floor beams. The two beams shown by dotted lines were placed there temporarily during erection to carry the erection traveller track. It will be noticed that the number of rivets over these beams is increased on account of the fact that the forward support of the traveller, while hoisting was sustained at these points.

The planking of the footwalks is continued between the truss members in order to take advantage of the extra width thus obtained.

The fence being next the road, and therefore liable to severe shocks, is a substantial structure 5 feet high; the rails are composed of 6" channels, the two upper ones being placed horizontally and the bottom one vertically. The posts are placed about 15 feet apart, those at the floor beams being firmly braced as shown, while those at the intermediate points are held by bracing extending out from the stringers. The fence is figured to resist a horizontal force of 2,000 lbs. applied at any point.

The anchorage at Pier No. 1, is shown on Plate 8. A steel grillage is placed 33 feet below the surface of the masonry which forms the counterweight and is composed of 2 plate girders running all the way across the pier, above which are supported several 15" eye beams built in the masonry. The main girders are supported by groups of 15" eye beams bearing on cast bolters through which the adjustable hangers pass. The four eye bars connecting the pins of the anchor arm descend through a well in the piers and are connected with the hangers by pins. Adjustment was considered advisable so that any small inaccuracy of placing the grillage could be rectified and also to put initial strain in the bars which would prevent any movement in the shoes of the anchor arm due to the considerable variation of the reactions.

The anchorage at pier No. 4 through the utilization of the dead load of the fixed span is much lighter, the eye bars at pier No. 1 being replaced by loop rods, and the counter-weight girders do not extend across the pier. In other respects the construction is similar.

The steel work at both piers is placed in wells which can be entered from below and inspected or painted. Ladder rungs are placed in the masonry to enable a workman to reach the whole length of the bars.

By giving fairly complete drawings, which should show the general construction of the bridge, it was not thought necessary to give a written description of the details, which are very similar to those used in everyday practice. The stresses are statically determined, and no ambiguity of stresses in the main members occur. Therefore, stress diagrams and other computations are omitted, since to put them in proper form for publication would entail a vast amount of labour, and would be of little interest to those conversant with the elements of bridge construction.

The principal part of the description is confined to explaining the methods and reasons for those details, which were somewhat difficult to design and were dealt with in a somewhat unusual manner. In this way it was thought that the most useful information could be presented to the members of the Society for discussion expending the time and space for its preparation to the greatest advantage and profit.

The method of erection, which is probably the most interesting feature, is not here touched upon, as it will be fully treated in a separate paper by the engineer who was in charge.

Thursday, 5th December.

E. MARCEAU, Vice-President, in the Chair.

The following donation to the Library was reported: —"Altitudes in Canada," by James White.

It was moved by Mr. W. J. Sproule, seconded by Mr. G. Janin, and carried—"That the Council is hereby recommended to obtain a list of Engineers practicing in the Province of Quebec who are not Members of this Society."

Discussion on paper by Mr. C. B. Smith on "Discharge of Sewage from Toronto Sewers in 1900," occupied the evening. (See Part I, page 128 of this volume).

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Thursday, 19th December.

JOHN KENNEDY, Past President, in the Chair.

**Paper No. 163.**

## CONSTRUCTION OF THE SUBSTRUCTURE OF THE ROYAL ALEXANDRA (INTERPROVINCIAL) BRIDGE AT OTTAWA, CANADA.

By GUY C. DUNN, M. CAN. SOC. C.E.

Since the Pontiac Pacific Junction Railway and the Ottawa and Gatineau, now the Ottawa Northern & Western Railway, have come under the control of practically the same management, it has been looked upon as a matter of absolute necessity that they should have an entrance of their own from the Province of Quebec into the City of Ottawa, in the Province of Ontario. After numerous surveys of the Ottawa River had been made, both above and below the Chaudiere Falls, it was finally decided to bring the two lines to a junction in the City of Hull and across to the Ottawa shore at Nepean Point, and construct an approach between the Rideau Canal and Major's Park, going under Dufferin Bridge, and cutting an opening through the abutment of the historical Sappers Bridge; thus making a connection with the Canada Atlantic system at the Central Station.

Several bridge surveys were made across the river at Nepean Point, but it was not until February, 1894, when a survey was made by the writer under Mr. W. Dale Harris, the then Chief Engineer of the two railways, that a line was laid down, which, with a few minor changes, was finally adopted. Owing to the heavy deposit of sawdust, slabs, etc., at the bottom of the Ottawa River, it was found necessary to employ a steam diamond drill as being the only practical means of getting a reliable profile of the bottom of the river. A short description of the manner in which this was done may be of interest.

After the centre line was marked off on the ice, every 50 feet across the river, and two lines were run parallel to this, one 100 feet up and one 100 feet down stream, borings were taken at every 50 feet on both these lines, and afterwards at intermediate points where considered necessary. In many cases, considerable difficulty was found in getting the drill through the refuse deposited by the mills, which consisted not only of sawdust, but of slabs and logs (some of the latter being of oak), and it was found necessary to abandon the diamond bit and use what is known as a blind bit, first filing the edge into rough teeth, and then sawing this through slabs, logs and sawdust until rock was reached, when the pipe would be withdrawn as gently as possible, the rough bit replaced by the diamond and boring continued into the bed rock. A core of clean limestone, $^{15}/_{16}$ inches, was obtained at each hole, and no soil of any kind was found on the rock. The sawdust deposit ran from shore to shore, the greatest depth found being 60 ft., with 20 ft. of water above it at winter level. Numerous explosions of sawdust caused by the accumulation of gases have occurred in the Ottawa River in this vicinity, in some cases strong enough to break the ice, upset row boats, etc., but no explosions of a serious nature occurred during the construction of our works. After this survey was made, which included cross sections and contours of the heavy side hill between Nepean Point and Dufferin Bridge, nothing more was done until January, 1898, when the services of Mr. G. H. Massy, were secured by the Companies, and the final location of the piers and lay out of the bridge were decided upon.

The bridge and approaches are over a mile and a third in length, and give the Railways a magnificent entry into the city of Ottawa. They consist of the Hull approach of 500 feet of earth embankment 15 feet high, 1,000 feet of timber trestle, averaging 20 feet in height, which crosses seven streets by steel girders on concrete piers, 690 feet of single track steel trestle on concrete pedestals, and 390 feet of double roadways.

The main bridge, from pier 6 to the south abutment, is 1495′ 9″ long, consisting of one cantilever span 555′ 9″, 2 arms 347′ each, one

truss 247′, one do., 140′, and two small shore spans on the south side respectively 27 and 32 feet long. From the south abutment on the Ontario side, at which point the waggon roads leave the bridge, the railway is built on rock embankment to St. Patrick St. Trestle, which is a three track steel trestle 270 feet long, the greater part being on a ten degree curve, and from the south end of the trestle to Dufferin Bridge, a distance of nearly half a mile, the track is on half rock cutting and half embankment, held in by a heavy dry masonry retaining wall, in some places 50 feet in height, and allowing for three tracks on a roadway 40 feet wide.

The contract for the bridge and approaches was let to Mr. H. J. Beemer, five of the water piers (1 to 5 inclusive) were let to Chas. H. Deans, and the contract for all the steel superstructure let to the Dominion Bridge Company, Limited, of Lachine.

The writer will avoid as much as possible the general details of construction which have been so often described, and will endeavour to deal with those characteristics of the work which may be of some interest to the profession, as being out of the ordinary run of bridge work, and in some cases peculiar to this undertaking, referring more particularly to the sinking of the caissons in the sawdust, the removal of the sawdust, the placing, sinking and filling of the caisson for pier No. 2 (which is one of the deepest concrete piers on the continent), and the successful boring from the top of the concrete in this pier to bed rock below, a distance of nearly 70 feet, and obtaining satisfactory core all the way through.

In laying out the river piers, owing to local conditions, it was considered advisable, by the Engineer in charge, not to use any base lines, but to rely entirely on intersections, and the work was laid out as follows:—A point was chosen on a wharf about 1,000 feet above the bridge and a permanent hub placed as shewn on sketch No. 13 at Point A.

The different piers were laid out on the ice, a steel ribbon over 1,000 feet in length being used, and a line was run in parallel to the centre line of the bridge, and twenty-five feet above it. The transit was sighted on the intersection of this line with the longitudinal centre line of the pier, the line produced to the shore (or as it happened in one case to a crib), and a permanent hub put in. This work, of course, had to be done quickly to avoid any movement of the ice, and checked over several times, an ordinary 6″ transit being used. The result was that all measurements on this structure came in practically exact.

Pier No. 5.—Construction was commenced early in February, 1898, the caisson for Pier No. 5 being the first to be sunk. Owing to the shallow water (21 feet), and there being only eight feet of sawdust at this point, no difficulty was experienced in cleaning off the

site and sinking the caisson to the rock bottom, which was ready to receive concrete on the 26th February. The depositing of concrete was commenced on the 28th February, in the manner described later, and completed on the 2nd of March.

Pier No. 4 was the next to be built. A derrick scow having been constructed and rigged up with a powerful hoisting engine and a clam shell dredge, a channel was cut through the ice to the site, through which, with much difficulty, owing to the anchor ice freezing to its bottom, the scow was hauled to the point where the pier had been located. The sawdust at this point was 17 feet deep, and the clam shell commenced its removal on the 27th February. The contractor had the bottom cleaned off, caisson in position, and concrete commenced on the 5th March. In removing this sawdust a considerable portion was deposited on the surrounding ice, and the residents of Hull carted away most of it to use as fuel; perhaps in the future the large deposit now resting on the bottom of the Ottawa River may be utilized for some such purpose. This caisson was filled in about four days. The caissons for piers Four and Five were ordinary bottomless ones of 12″ x 12″ hemlock. Masonry was commenced on piers 1, 4 and 5 (the former being a shore pier) immediately after the completion of the concrete filling.

Pier No. 3.—On the 26th July dredging out sawdust was commenced at Pier No. 3. The depth of deposit at this pier was 20 feet, with 22 feet of water above it, the clam shell removing the sawdust very slowly, owing to the slabs getting crossways in the clam and allowing the sawdust to run out. The excavated material was deposited into scows and towed to shore, where it was dumped. During this excavation a number of large logs were hauled out and a large sized anchor with piece of chain attached, the links of which were so badly corroded that it fell apart. The excavation was made considerably larger than the area of the caisson, the sides having a slope of probably about 1 to 1. The sawdust itself formed a fairly compact mass, as the slabs tied it together and assisted to keep it from falling in. On the 12th August, the preliminary cleaning off of the foundation rock was completed and the caisson towed into place. This caisson and the one for pier No. 2 were of similar design. They were built of 12″ x 12″ hemlock, braced with 4 heavy timbers over every four courses and tied into the sides. Owing to the depth of water at these two piers (about 46 feet at pier No. 3 and 74 feet at pier No. 2, summer level), the caissons were designed in such a manner as to allow all weight necessary to sink them to be placed at the bottom, and at the outside, therefore not robbing the concrete of any of its legitimate area inside the caisson. As will be

seen by the plan, the upper part of the caisson was built on the same batter as the masonry, 1 in 24, but when it reached a point about 15 feet from the bottom, the batter was sufficiently increased to allow a wall to be built from the cutting edge up all around the caisson and tied into the main wall, thereby forming a pocket capable of holding enough sinking material to lower the caisson to the desired depth. The plans for these caissons were designed by Mr. Lee Treadwell, the contractor's Engineer. After the caisson had been placed approximately on centre, sufficient sand and broken stone were deposited into the sinking pocket to lower it until cutting edge was almost touching bottom. Divers were then sent down to remove any remaining sawdust, which was sent up in a large iron bucket to the surface. This took a considerable time, the divers complaining greatly of the heat caused by the accumulation of sawdust around the caisson; the caisson was then placed exactly on centre and lowered to the bottom. The bottom edge having been scored when constructed to approximately fit the bed rock, no trouble was experienced in placing this caisson, as there was no current at this point and the bed rock at this pier as well as at piers 4 and 5 did not need further levelling. After the divers had packed any holes and crevices found around the bottom of the caisson, the Company's inspector was sent down for a final examination, and on his report being satisfactory, depositing the concrete was commenced on the 25th August, and continued night and day, until completion on the 31st day of the same month. The concrete for this pier as well as at piers 4 and 5 was composed of one of cement, one of sand and about five of broken stone. The broken stone was not screened, as after testing the stone chippings by using them instead of sand in briquettes, the result was very satisfactory, and it was decided to allow their use in the concrete.

The concrete at piers 4 and 5 was mixed by hand and that at pier 3 was mixed by a Sooysmith mixer which did excellent work and gave entire satisfaction. It is a horizontal cylindrical machine about 14 feet long, having a shaft running longitudinally through the centre, to which was attached a number of paddles. It was placed on a scow alongside the caisson. The cement, sand and stone were supplied by carriers on endless chains and the water obtained from a tap controlled by a man. The cement and sand were the first to enter and got thoroughly mixed before they reached the point where the stone and water entered almost simultaneously. Everything then got a thorough mixing and was finally ejected from the end of the cylinder into the depositing kibble. As the carriers were set at their proper proportion before the machine was started, and all worked from the same gearing, there was no measuring required; all the inspector had to do was to see that the carriers were kept full.

Pier No. 2.—The construction of the caisson for pier No. 2 (see plan No. 15) was commenced on the Hull shore. The caisson was launched and then towed over to the quiet water below Nepean Point, where it was completed. This caisson, from top to bottom, was about 76 feet high, and as previously stated was a similar design to that of No. 3. The clam shell commenced excavating the sawdust at pier No. 2 on the 2nd September, 1898, and the process of removal was similar to that employed at Pier 4; owing, however, to there being a more swift current, a larger area had to be excavated before the caisson was placed. The sawdust at this point was 15 feet deep, with 57 feet of water over it. Two scows 80′ x 20′ each were now placed, one on each side of the caisson; a truss built from one scow to the other at each end (see plan No. 15), and the caisson suspended from them by heavy cables, sufficient ballast having been put into the sinking pocket to keep it steady. On the 16th of September they were towed into position, the current at this point running at about 3 miles an hour. Cables were then stretched from the scows to rock bolts, and to pier No. 1 on the Ontario shore, to two heavy ship anchors on the river side, and up stream to a large Chinese anchor, the cable from this anchor entering the bow of the caisson some distance below water level. Although all the anchors were bearing on sawdust debris, no dragging occurred.

The caisson was then lowered almost to the bottom and divers sent down to remove the balance of the sawdust. They found the shore slope of the rock to be very smooth and to extend some distance into the caisson. The contractor was therefore instructed to blast this section from one end of the caisson to the other. A steam drill was brought into requisition and a line of holes drilled about 3 feet deep and 5 feet apart. A diver was sent down to load them and they were fired by the battery from the top. The loose stone was then removed and the caisson placed in position, being almost to an inch on centre both ways, but owing to its extreme height it was difficult to ascertain as to its being plumb. This was, however, checked in three different ways; firstly, by a level across the caisson, and secondly, by its batter, by running a straight edge down the side. Owing to the great height of the caisson—nearly 80 courses of timber—these tests were not considered sufficient, and as a third test a plumb bob was obtained, weighing about 50 lbs., and was lowered 50 feet from the inside edge of the caisson, first on one side and then on the other. A diver was sent down with a stick, with which he measured from the sides of the caisson to the cord, marking the place on the stick; repeated trials gave the same result, and the caisson was found to be practically perpendicular. The divers were once more sent down to pack any crevices at the bottom of the caissons with bags of neat cement.

The diving at this pier was a very interesting feature of the work, and the men worked under great difficulties, owing to the immense pressure. Half an hour as a rule was the length of time they could stay down, although on one occasion one man remained down for two and a quarter hours. The water was very dark and nothing could be seen after they had descended twenty feet below the surface. After the Company's diver had made an entirely satisfactory report, the contractor received instructions to commence the deposit of the concrete, which he did on the 9th of October. The concrete for the lower twenty-five feet of this pier was mixed with the mixer and the specifications were altered to 1 of cement, 2 of sand, and about 4 of broken stone. Work was continued until the 17th of October, when 24 feet in depth or practically one-third of the concrete portion of the pier had been deposited. Just at this time an unfortunate accident happened to a large bridge under construction, killing a number of men and causing several sensational articles to be published in the local papers regarding bridge work in general and concrete in particular, and, by orders from the Dominion Government, work was suspended on this pier, the Government Engineers holding the Companies to a certain clause in their specifications, which stated that the caisson might be pumped out and the balance of the concrete deposited dry. To do this with the caisson which had been constructed was impossible, and the cost of adapting it for and obtaining a compressed air plant was prohibitive, or at least was not considered to be warranted, as the concrete and the methods of depositing were believed to be in every particular satisfactory. A test of the concrete already deposited by means of a diamond drill was then suggested, but it was not until the 10th of March, 1899, that permission was obtained from the Government to proceed with the work conditional to satisfactory core being produced through the 25 feet of concrete to the bed rock. The same drill was obtained that was used on the survey in 1894, and operations were commenced, but, owing to the depth of water which had to be drilled through, and, although a diver was sent down to steady the drill rods which were old and shaky, it was found impossible on account of the great vibration to obtain satisfactory results. A request was then made to the Government for permission to deposit the remainder of the concrete to its calculated height, conditional to our giving them a satisfactory core from top to bottom. This was finally agreed to, and concrete again deposited after first washing off the top of the old deposit as well as possible with a steam pump. Owing to the test that had to be made and on account of not being able to obtain any facts in regard to any previous borings in concrete, the proportions of the concrete in the last deposit were increased to 1 of cement, 1 of sand, and what stone it would take, and the caisson was finally filled on the 1st of April of the

same year. This last deposit was mixed by hand, the weather being too cold to allow the use of the mixer. The platforms were on the Hull shore and the concrete shovelled into the kibbles, which were drawn to the pier on sleighs. The sand and water were slightly warmed, very little salt being used, and the temperature of the concrete, when placed in the kibbles, averaged 55 to 58 degrees Fah., and did not vary before being deposited under water. The concrete was allowed to set until the 19th of August.

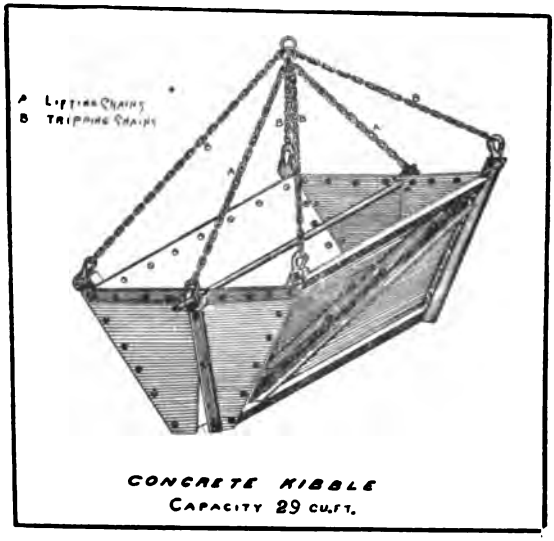
During the deposit of the concrete in the lower portion of this caisson a diver was sent down to report as to whether the concrete was leveling itself off or not, as an opinion was expressed that it should be raked over or shoved with a hoe to the sides and into the nose at the bow, but he reported that it was of its own accord filling up to the sides of the caisson most satisfactorily, and did not require any assistance.

When the caisson was un-watered the best diamond drill to be found was secured and started in the concrete. The first bit used gave a core of 2⅛″ diameter, and satisfactory results were obtained from the start. The hole was run down 40 feet when, owing to the wall giving so much trouble and delay on account of a lot of loose stone, broken core and chippings which had scraped off, it was considered better to start a new hole than to take the time to clean this one out, and hole No. 2, which was carried through the bed rock, was commenced. No difficulty was experienced until a depth near the bottom of the pier was reached, when the vibration of the drill became so great that collars had to be attached to the tubing, and when near the bottom the bit was changed for a smaller one, giving a core of 1⅜″ dia., which was used until bed rock was reached and a piece brought up. This core was obtained in pieces from about two to six inches long and is now in the possession of the Dominion Government, who had an inspector on all the work done on this pier. In this boring the drill went through four different brands of cement, and no appreciable difference could be noticed in the core obtained as we passed from one deposit to another. The diamond drill was also run into the foundation of Pier No. 4 for about 12 feet, and excellent core obtained.

The concrete in all the piers was deposited through the water until it reached a point about 3 feet below the extreme low water mark, and after it had sufficiently set, the water was pumped out and the concrete levelled up to within two feet of low water ready for the masonry.

The concrete kibble used for depositing the concrete is shown in the sketch below and was found very satisfactory. Owing to its wedge shape it created very little disturbance in going through the water, and had a capacity of 29 cub. feet. The doors opened from

the bottom, and every precaution was taken to guard against the kibble being tripped before reaching bottom. A canvas cover was used, and tied securely to each bucket before it was lowered.




During the progress of the work on pier No. 2, the following tests were made when the first deposit of concrete was being put in. A kibble full was put down in the caisson at the usual speed to a depth of about sixty feet and then raised slowly to the surface. Owing to the flat surface of the concrete at the top of the bucket the water offered a strong resistance to motion through it and gave a very severe test for wash. On the bucket reaching the surface, very little wash was noticed and two cement barrels were filled from the top and put under water to set. Two weeks later a diamond drill was run through them in several places, and gave excellent core. When the caisson was nearly completed one more test for wash was made. A large wooden box, having a capacity of over two cub. yards, was made and sunk in the caisson on top of the concrete deposit in about eight feet of water. A kibble was then filled with concrete and let down through the ice into the stream outside the caisson to a depth of over 50 feet. It was raised with the intention of lowering it into the box, but, owing to a mis-calculation, the latter had been made too small, and it was necessary to trip it above the box, letting the concrete sift through 3 feet of water; this was repeated with another bucketful, which filled the box. Owing to the severity of the test it was not

thought that the sample would be of any use, but when the caisson was de-watered it was found in a very satisfactory state, and on the diamond drill being run through it, although no core was obtained, the wall stood hard and firm.

The following brands of Portland cement were used in this pier, viz., Star, Hercules, Alsen and Mannheimer, a sample of each being tested by McGill College, and continuous tests being made by ourselves all through the progress of the work. The sand used was good, clean river sand, which was obtained from under the surface of the water near the Hull shore, about two miles below the bridge.

The Company had a number of inspectors on the work day and night, their chief duties being to check quantities used in the concrete and superintend the mixing of the same; to see that the concrete bucket was lowered very gently until it was under the surface of the water; and to also see that it was not tripped until it reached bottom. The Government also had an inspector on all the work on No. 2 pier, and on most of the other important work. During the winter of 1898-1899, grave fears were entertained for the safety of caisson No. 2, as it had then only 24 feet of concrete at the bottom, and there remained over 50 feet standing up in the river, a hollow box, exposed to local ice shoves and to the steady pressure of the anchor ice which at this point we found to be over 30 feet deep. To as far as possible counteract any pressure, the contractor was instructed to put extra bracing inside the top of the caisson and to keep the surface ice out for a distance of several feet around the caisson. This he did, and no ill effects were experienced; but had the balance of the concrete not been deposited in April, the spring ice floating out would most assuredly have destroyed the caisson. The filling up was only completed on the 1st of April. On the 19th of the same month the ice was moving out of the river, and one block from the bay at the foot of the locks with an area of over 30,000 square feet of surface ice about two feet thick, and anchor ice below that anywhere from 10 to 20 feet thick struck the top of the caisson and rested there for several days; efforts were made to blast it, but dynamite had no effect, and, when it finally broke away, it was found that a few of the top timbers and some of the bracing of the caisson were slightly injured, but otherwise no harm was done. This in itself was a severe test on the pier. Another severe test received by this pier was during the erection of the superstructure; the south half of the cantilever span was swinging out a distance of over 270 feet for upwards of three months and was during that time exposed to some gales blowing over 60 miles per hour.

All the upper portions of the piers are solid masonry, no concrete backing being used, and were built under the usual specifications for first class masonry, the bridge seats and copings being

hammer dressed, and the remainder rock face. The limestone used for piers Nos. 1, 2 and 3 came from Eganville, Ont., except the copings of two or three, which were obtained from Crookston, as were the copings for piers 4 and 5. The balance of these latter two piers were built of stone from Rockland, Ont. The stone for pier No. 6 was brought from the Terrebonne quarries. Concrete was used for all the trestle pedestals, also for the abutments of all bridges over streets in Hull.

From the south end of the bridge to Central Station, three tracks were laid of 75 lbs. steel rails, with crushed stone ballast, the two outside ones now being operated by an electric service running from Ottawa to Aylmer across the bridge on the highway portion, and the centre one is used by the trains of the Ottawa Northern and Western Railway, and in a week will also carry the trains of the Pontiac Pacific Junction Railway. To make a connection with the tracks of the Canada Atlantic Railway at Central Station, as the writer previously stated, it was necessary to cut out a portion of the abutment of the Sappers Bridge, which is a stone arch over the Rideau Canal, and was built by the Royal Engineers in 1827; the abutment was 24 feet thick, the walls being anywhere from two to 4 feet thick and on the south side the top of the wall was about 2 feet thicker than the bottom. The lime used had deteriorated until it apparently had no consistency, but the masonry held together in a most remarkable way. Earth filling had been chiefly used between the walls.

The superstructure of the main bridge is being dealt with in a paper by a member of our Society, and the writer will refrain from touching on it, except to state that the bridge is free to vehicles and pedestrians forever and that the Company keeps the waggon roads in repair for thirty years. Before being opened to the public the bridge and approaches were inspected and found entirely satisfactory by the following officials: R. C. Douglas, Bridge Engineer, and E. V. Johnson, Inspecting Engineer for the Dominion Government; Louis A. Vallée, Director of Railways for the Province of Quebec; Robert McCallum, Inspecting Engineer for the Province of Ontario; and Newton J. Ker, City Engineer of Ottawa, Ontario.

The first locomotive crossed the bridge on the 12th December, 1900. The roadway portion was opened to the public on the 5th March, 1901, and the first passenger train to cross was the Ottawa Northern & Western Railway express, on the 22nd April, 1901. This undertaking as a whole is a monument to the energy of Mr. H. J. Beemer, the original promoter and, later contractor, who, practically unaided, and with seemingly unsurmountable obstacles in his-

way, has finally brought the work to completion. The permanent Engineers' staff was as follows:

Guy C. Dunn, Chief Engineer.
F. A. Hibbard, Resident Engineer.
A. W. H. Stimpson, Ass't. Engineer and Draughtsman.
C. E. Fouse, Inspector Superstructure (Pittsburgh Testing Laboratory).

The following is a summary of a few of the quantities:

| | Sawdust. | Concrete |
|---|---|---|
| Pier No. 2..... | 1,000 cu. yds. | 2,579 cu. yds. |
| Pier No. 3..... | 800 " | 1,462 " |
| Pier No. 4..... | 370 " | 728 " |
| Pier No. 5..... | 300 " | 394 " |

## DISCUSSION.

MR. C. E. W. DODWELL remarked that the Alexandra Bridge at Ottawa, of which the substructure was described in Mr. Dunn's paper, would rank as one of the notable bridges of the country in length, cost and importance, if not in boldness of design and difficulty of construction. The railway companies, to whose enterprise and far-sightedness its projection was due, were to be congratulated on its completion. The only distinctive feature of especial interest that distinguished this work from many others of a mainly similar character was the remarkable depth of sawdust through which the pier foundations were carried to rock bottom. This, while probably greater than in any other river in Canada, owing to the magnitude and extent of the lumbering industries of the Ottawa, was not peculiar to that stream. In the Maritime Provinces there was more or less sawdust in every stream big enough to drive a mill or float a log, and for many years (in some cases nearly a hundred) not only sawdust, slabs and the refuse from sawmills, but, in Nova Scotia at any rate, the pulverized tailings from quartz crushers, had been thrown, not only with impunity, but with an absolute disregard of consequences, into streams that formerly teemed with trout, salmon and other fish. In the La Have River, where he had made a survey four or five years ago with a view of some dredging for the widening of the ship channel, there was found sawdust of a thickness of nearly 20 feet in depth for several miles. The result of this wholesale, reckless pollution of the streams was that in the most beautiful of them trout and salmon were now rarely found. It might perhaps be urged in defence of this time-honoured mode of disposing of mill refuse that the lumber interests of the country were of more importance and value than the river fisheries; but when the sawdust and slabs could so easily, and with such an insignificant additional outlay, have been burned or otherwise so disposed of as not to be a nuisance and positive detriment, it was a great pity that proper legislative restrictions were not made and enforced in the earlier days of the lumbering industry of the country. At this date most of the sawmills in Nova Scotia and, he believed, in New Brunswick also, burned their refuse either as fuel or in specially constructed furnaces or incinerators; but the mischief to the streams was done, and the lordly salmon sought purer and sweeter spawning grounds. It was to be hoped that some day an inventive genius would discover a hidden value in sawdust. For such there would await a princely fortune. The mode of excavating the sawdust for the pier found-

ations seemed to have been simple and effective, and the nature of the work to be done peculiarly suited to the clam-shell form of bridge. It was fortunate for the contractor that between the sawdust and the rock there was not found a stratum of gravel or clay. The form of kibble for depositing the concrete did not appear to him to be an improvement on that adopted at the Canadian Pacific bridge over the St. Lawrence at Lachine, which was a square iron box, holding 2 cubic yards, with a closing top hinged in two leaves and a similar bottom tripping by means of a simple latch. This construction caused the minimum disturbance to the concrete, and its operation was everything that could be desired. The author's paper had the same defect that characterized nearly every paper of a similar nature that had ever been published in the Society's Transactions, namely, the conspicuous omission of all mention of cost and prices. It was comprehensible that there might be instances in which, for business or other reasons, there would be objections to the publication of the details of the cost of certain works, but, in many cases, authors of papers would not find these objections insurmountable, and they should consider the tremendously added value that would attach to their papers by making the subject of cost a prominent feature. He hoped that in his reply to the discussion the author would add an appendix to his paper giving particulars both of cost and quantities.

Mr. G. H. Massy observed that, so far as he was aware, the first instance in which bottomless caissons filled with concrete were used was in the construction of some of the bridges on the Intercolonial Railway about thirty years ago. Since then, this class of foundation was adopted in the construction of the Chaudiere Bridge at Ottawa in 1879, on the Lachine Bridge in 1886, and at the International Bridge at Sault Ste. Marie in 1889, together with many other cases too numerous to mention, thus giving a continuous practical test of this class of work extending over thirty years, and in every case, so far as he was aware, the result had been satisfactory. As in all work which depended upon the excellence of the concrete, the greatest care was required in the choice of materials, proportions, mixing and depositing. Allowance had also to be made for loss of cement due to the washing action of the water during the process of depositing, especially where there was any current; and also, it should be remembered, that once the concrete was deposited it must be neither rammed, leveled nor otherwise disturbed, as the least agitation separated the cement. Ramming would probably add one-third to the strength of concrete, but it must be dispensed with in this class of work. Much had been written and said about concrete, but still it was an open question as to what the proper proportions should be for certain classes of work. The strongest concrete was probably produced when the mortar was rich in cement and no more mortar

used than was sufficient to fill the voids in the stone. Another source of danger to be guarded against was the deteriorating effects of water forced through very porous concrete, which he was inclined to think might cause trouble. More experiments were required on these points. It would be generally admitted that machine mixing when well done was superior to mixing by hand; but with some continuous mixers, especially when starting and stopping, the concrete produced was either too wet or too dry, or had not the proper proportion of ingredients. The author appeared to be fortunate in securing the services of such a good continuous mixer as that used in the Alexandra Bridge.

The AUTHOR, replying to the discussions on his paper, agreed with Mr. Massy in every particular, unless, perhaps, as to concrete losing one-third of its strength by not being rammed. In reply to Mr. Dodwell's criticism, he observed that from the experience he had had in depositing concrete he considered the V-shaped kibble, as used on the substructure of the Interprovincial Bridge, an improvement (especially in deep waterwork) on the square bucket usually adopted, as, on account of being wedge-shaped, it cut through the water with much less disturbance than the square bottom, and when opened, while resting on the bottom, the concrete was deposited on the foundation rock without having to pass through a foot and a half or two feet of water, as necessitated by the square bucket. As regards the omission of costs and prices in his paper, he fully appreciated the additional value of a paper in which such figures could be given; but the engineer in charge of work was placed in such a delicate position of trust, both in the interests of the company he was working for and the contractor who was doing the work, that, without the joint permission of both parties concerned, it was impossible that such figures could be given.

**Paper No. 164.**

# ERECTION OF THE ALEXANDRA BRIDGE.

By H. D. Bush, M. Can. Soc. C.E.

This bridge crossing the Ottawa River between Ottawa, in the Province of Ontario, and Hull, in the Province of Quebec, was called, during construction, the Interprovincial Bridge, at Ottawa.

During the visit of the Duke and Duchess of York to Canada, in 1901, the bridge was formally opened, and named after the present English Queen.

The method of building the substructure and the design of the superstructure have already been described in the papers of Guy C. Dunn, Member, and F. P. Shearwood, Asso. Member, of this Society.

The profile shown on Plate 16 gives the numbering of the piers from 1 on the Ottawa to 6 on the Hull shore. The spans will be designated by the numbers of the piers they connect, as 2-3 for the cantilever span, etc.

Somewhat unusual conditions were found here. From the Ottawa shore the bed rock slopes abruptly to a depth of seventy feet in a distance of about 200 feet. From this point to the Hull shore the bed rock is covered with a deposit of saw-dust, mill refuse, etc., in one place to a depth of 50 feet.

The general method of erection was decided on by the engineers of the Dominion Bridge Co., Ltd., when the contract for the superstructure was awarded. A central cantilever span of 555 feet 9 inches was designed, and it was planned to erect the two cantilever anchor arms each 247 feet and one other span of the same length, on falsework, to be supported on four scows carrying timber trusses for distributing the load, the scows being connected by the trusses, so that the whole structure could be floated together from span to span.

The writer's connection with the work began in July, 1899. At that time the scows were being built at Lachine, the drawings for same having been made immediately after the contract was let. The writer then, under the general direction of Mr. Phelps Johnson, M. Can. Soc. C.E., Manager of the Dominion Bridge Co., Ltd., made designs for the remaining features of the erection plant, timber, trusses, derricks, traveler, etc., and, later, went to Ottawa, remaining there during the construction of the bridge.

The bridge was built for what is now called the Ottawa, Northern & Western Railway Co., and is arranged to carry one steam

railway track between trusses 24 feet centre to centre. The spaces on each side of the track between the trusses are used for foot walks. Outside the trusses, on each side, is a roadway 21 feet wide, supported on cantilever floor beam brackets, carrying an electric railway track close to the trusses, and leaving room for the vehicles outside of the same. This makes a total width of bridge of 66 feet.

The metal weight of the cantilever structure alone was approximately 5,000,000 lbs.

The outer floor beam brackets and stringers were not placed in position on any span until that span had been fully connected and become self-sustaining. The weight of metal in one of the 247 foot spans, as erected on the falsework, exclusive of the roadways, pier members, etc., was 910,000 lbs. The weight, exclusive of traveler, supported by the scows during erection, was about 1,400,000 lbs., made up as follows:—

| | | |
|---|---|---|
| Metal work.. .. .. .. .. .. | 910,000 lbs. | |
| Howe Truss and Falsework on same.. .. .. .. .. .. | 308,000 lbs. | |
| Blocking.. .. .. .. .. .. .. | 42,000 lbs. | |
| I-beams and blocking under Howe Truss.. .. .. .. .. | 140,000 lbs. | 1,400,000 lbs. |

As there were four scows employed, this gave a load of 350,000 lbs. per scow. In addition to this, each scow in turn had to support the weight of the traveler (except as it might be partly distributed by the Howe truss), amounting to about 140,000 lbs.

The maximum load for each scow was taken at 450,000 lbs., or 112,500 lbs. at each of the four points of bearing on the scows.

The scows were especially designed for the concentrated load to be placed on them during erection. Each scow contained two longitudinal steel trusses, which were 22 feet 6 inches apart between centres, 6 feet 3 inches high, and divided into panels of 6 feet 3 inches.

Opposite panel points of the bottom, chords were connected with 15″ —42″ rolled I-beams, under which were 4 inch by 12 inch longitudinal timbers, to which the 3 inch bottom planks were spiked.

Timber posts above the floor beams supported timber deck beams and planking.

The construction of the scows is shown by the drawing, plate No. 16.

The scows were 100 feet long, 26 feet wide and 8 feet deep outside measurements.

Each scow was estimated to weigh 150,000 lbs., and, when empty, to have a draft of 13⅜ inches.

The estimated displacement of one scow was as follows:—

| | | | Less weight of scow. | Net. capacity for load. |
|---|---|---|---|---|
| | Lbs. | Lbs. | Lbs. | Lbs. |
| For 2 feet draft | 277,000 | 277,000 | 150,000 | 127,000 |
| For 3 feet draft add | 149,500 | 426,500 | " | 276,500 |
| For 4 feet draft " | 154,750 | 581,250 | " | 431,250 |
| For 5 feet draft " | 158,500 | 739,750 | " | 589,750 |
| For 6 feet draft " | 161,000 | 900,750 | " | 750,750 |

The maximum estimated load of 450,000 lbs. per scow, would therefore, correspond to a draft of 4 feet 1½ inches, and the load of 350,000 lbs. (not including the traveler), a draft of about 3 feet 6 inches.

The contractors for the superstructure had been given to understand that there was very little change in the level of the Ottawa River at this point during the months of October to March, when it was proposed to erect spans 6-5, 5-4 and 4-3. In fact, the writer was instructed to work on the assumption that the average height of water would be at elev. 94 above the bridge datum, and that it might fall to 93 or rise to 95.

During the year 1899 it was difficult to get early delivery of steel from the rolling mills. For this reason the manufacture of the bridge was so delayed that the erection of the steelwork, starting at pier 6, could not be begun until December, 1899. The erection of span 5-4, the first one on which the floating falsework was used, began January 12, instead of at least two months earlier, as originally planned. This delay gave rise to unexpected difficulties in moving the scows through the ice, as will be described later.

It was also learned that there was more variation in the level of the river than had been expected. From the 13th to the 17th of December the river rose from elevation 94.5 to elevation 98.1. This occurrence, three weeks after the writer's arrival at Ottawa, prompted him to examine the records of water levels. Mr. A. T. Phillips, M. Can. Soc. C.E., kindly gave access to the records, in the office of the Rideau Canal, of the gauge at Lock No. 1, in the bay just above the Ottawa end of the bridge. The elevation of the bottom of this gauge is 85.92 feet above the bridge datum. The readings of the gauge from 1884 to 1899 were copied and a table made, adding each reading to 85.92 giving the height of water above the bridge datum. From this table the chart on Plate No. 18 was prepared. This chart has been made complete to December 31, 1901, so as to include the extremely low water in the fall of that year, and it is here given, not only for its connection with the bridge erection, but in the belief that it will be found of value to engineers called

upon to study the flow of the Ottawa River with reference to power development.

It will be seen from this chart that a sharp rise in November and December is a common occurrence, but that outside of these months the variation in height in any one year in either September, October, January or February is about 3 feet.

It was proposed to take care of most of the variation in the water level by pumping water into or out of the scows, starting work with water enough in them to give 5 feet draft. Pumping in more if the river rose, and pumping out if the river fell, and, in any case, pumping out water to correspond to the weight of metal added as erection progressed.

There should have been about 2 feet of removable blocking between scows and truss for erection of spans 5-4 and 4-3. Span 5-4 could not have been placed on the piers at the time of the December rise, for instance, without this, although in January it was not needed.

Two upright boilers, of about 20 H.P. each, were set up on one of the inner scows, and 2 inches steam pipe run from the same to the other scows, with branches to six steam syphons: 4 (1 on each scow) used for pumping water out of the scows, and 2 (1 suspended between each pair of scows, with spouts running to either scow) for pumping water into the scows. The syphons had 6 inch suction pipes, and were used because they happened to be in stock in the Dominion Co.'s erection plant. The steam pipes were protected with cellular asbestos covering and enclosed in a wooden box.

The trusses over the scows were of the Howe Truss type, 20 feet high. The panels of the bridge were each 30 feet 10½ inches. The Howe Truss panels were one-half of this, or 15 feet 5¼ inches. The angle blocks were of oak, with 10 inch faces. The chords were made 33 inches wide of 3 inch by 12 inch plank, dressed to exact thickness, in 8 continuous leaves (4 spliced in each panel, making the planks 30 feet 10½ inches long), and three packing and splicing pieces, also 3 inches by 12 inches in each panel, 15 feet long, between the ends of which were spaces, through which passed the rods. There were three of these rods at each panel point, and they were all 1⅜ inches, except at the ends, where they were 1½ inches square. The planks were well bolted together with 28¾ inch bolts per panel. The braces were composed of two pieces, 10 inches by 11 inches, and the counters of 1 piece 10 inches by 11 inches.

The general arrangement of Howe Truss and scows is shown in plate No. 16. The alternate top chord panel points of each Howe Truss were assumed loaded with 90,000 lbs. each. The trusses were figured to carry this load if any one of the scows sunk by accidental dropping of material from above, assuming, however, that the ex-

treme ends of the trusses were supported. The concrete foundations of the piers provided a good offset outside the stonework, on which timber bents were placed. Blocking was kept on these bents under the ends of the Howe Truss as a safeguard, and to increase the stability of the structure against tilting by wind.

The loads being brought over the scows at panel points 30 feet 10½ inches apart, and the steel trusses in the scows being 22 feet 6 inches C to C, five 20 inch I-beams, 32 feet long, were placed across the scows under each truss to prevent bending stresses in the chords. These were clamped together with wooden blocks and bolts to avoid punching. Oak blocks, 8 inches by 10 inches, were placed between these beams, and bottom chords of Howe Truss close to truss rods.

On top of the Howe Trusses were 7 bents of falsework, 4-20 feet high, and 3-15 feet, 10 feet and 5 feet high respectively, to fit under the bottom chords of anchor arm spans. Temporary bents were placed on top of the short bents to bring the top of falsework level for span 5-4.

The falsework bents were each supported on two 20 inch beams extending across from truss to truss, placed on the top chords close to bearing plates of truss rods.

After the scows had been launched at Lachine, the Howe Trusses and top falsework and the traveler to be used in erecting the bridge were framed in the shop yard and loaded on the scows, together with five 12 ton stiff-leg derricks and the I-beams, anchors, chains, etc., to be used. Five scows had been built, the fifth one to be used as a derrick scow. The scows with their loads left Lachine in November, 1899, just before the close of navigation. The material was unloaded at the Hull end of the bridge.

The scows were placed in position between piers 5-4, and the Howe Truss and top falsework erected on the same with the derrick scow. Three of the 12 ton derricks were set up along the Hull approach, one with a 72 foot boom, close to pier 6, to lift material up to load on lorries on the bridge track.

Arrangements had been made to carry the steelwork from Lachine to Hull over the Canadian Pacific Railway. From the Hull station, about 1½ miles from the bridge site, the cars were brought down at night over the tracks of the Hull Electric Railway Co., using an electric locomotive. Much of the material, on account of its length, had to be loaded on two cars, and it may be of interest to note that the cars passed over one curve of 80 feet radius in the Hull Electric Co.'s tracks, and one of 100 feet radius in the Dominion Bridge Co.'s spur track without difficulty.

Steel began to arrive at Hull, December 10, 1899, and the erection of span 6-5 began December 20, 1899. This span (6-5) was erected on ordinary falsework, two bents on the ground and two

bents supported on 61 feet plate girders belonging to the Hull approach to the bridge, which were placed to span from an old crib wharf to a bent set on the concrete foundation of pier 5.—This span was erected by a simple movable bent. In all the spans the vertical posts, after being bolted to the deep floor beams, "stood alone," which fact greatly simplified the erection.

As soon as span 6-5 was completed, January 3, 1900, the erection of the traveler was begun. This consisted of a rectangular "inside" tower supporting top trusses, which overhung the tower at each side in the plane of the bridge trusses, and also overhung one and one-half panel lengths, or 46 feet 3¾ inches in front of the tower for the cantilever work.

The tower was erected on span 6-5, and the two top trusses put together complete on the ground, one on each side of the bridge, and hoisted up one at a time, as shown in Photo No. 1, January 10, 1900.

The traveler tower was made 18 feet 6 inches wide and 28 feet long, C to C of posts. This width just gave clearance on each side inside the bridge trusses. The length of the tower was made less than the bridge panel length, so that, with centre of forward posts at a panel point, top lateral and sway bracing could be bolted in place just behind the traveler.

The rear posts were 12 inches by 12 inches. The forward posts were each made up of three leaves of 7 inch by 12 inch timber bolted together to break joints. The two outer leaves were 63 feet high. The centre one extended up 30 feet higher, supporting a 7 inch by 12 inch cross timber, 22 feet long, and, over that, a 15 inch 50 lbs. I-beam, 26 feet long, to which the tackles for lifting the top trusses were attached.

The tower was braced on four sides by timber horizontal struts and diagonal adjustable rods, except in the bottom panels of the transverse bracing.

About 18 feet above the track there was a working platform, which extended 20 feet back of the rear posts. Just above this platform, between forward posts, the timber strut of the wind bracing was replaced by a 14 inch by 14 inch cross timber, intended as a beam to hold the snatch blocks through which the lines from the erecting tackles led to the hoisting engines. From the centre of this beam plank braces to the bottom of the posts completed the transverse wind bracing. A similar, but lighter construction, was used between rear posts. This arrangement left a clear space for a central track, on which material was brought forward as required.

The various features of construction, and the assumed wind pressures and resulting stresses are shown on Plate No. 17.

Two 24 inch I-beams were placed across the traveler over forward and rear posts, those in front resting on the outer 7 inches by

12 inch sticks. These beams projected on each side far enough to give support to the top trusses, which were placed 24 feet apart, centre to centre.

The anchor rods, from the back of the top trusses, were fastened to the sills of the traveler. The weight of the booms, overhanging in front, together with tackle, etc., was only slightly overbalanced by the weight of the rear portion of the traveler, including the empty engine platform. The estimated weight of the foot of each rear post was 3,000 lbs., and at the foot of each forward post 57,000 lbs. The weight of the boiler, engine, etc., gave sufficient excess of weight on rear posts to make the traveler safe when moving. The traveler ran on eight 24 inch double flanged wheels, two under each corner, the weight on each forward wheel being approximately 30,000 lbs. Double lines of rails, on each side of the bridge, were supported by ties laid on double stringers of 20 inch I-beams belonging to the outer roadways. These were bolted between the floor beam webs, and removed when convenient after the traveler had passed over them.

When the traveler was in use, short anchor rods fastened rear ends of sills down to the floor beam underneath, and iron wedges were placed under forward truck castings to prevent excessive loading on forward wheels and axles.

A study of all the possible combinations of members, during the assembling of the cantilever spans, was made to determine the maximum loads on overhanging beams before top trusses were designed. It was found that an anchorage re-action of 73,000 lbs. would be required at the rear end of each top truss.

The two top trusses were braced horizontally with timber struts and diagonal rods, the cross struts arranged to give support for tackles in addition to that provided by two inch U bolts.

There were two engines, placed tandem, on the working platform, each having two 7 inch by 12 inch cylinders, and operating two shafts with a spool at each end, or eight spools for the two engines. A 20 horse power upright boiler furnished steam.

The heaviest single bridge members weighed 12 tons each. The tackles for lifting these were of 2 inch diameter Manilla rope, rove in 16 inch triple blocks.

The traveler, though designed especially for cantilever work, handled the three spans on the falsework very well, and was especially convenient in laying the floor system ahead of its travel.

In order to load the falsework gradually, the traveler advanced from pier to pier assembling the floor system, lower chords and vertical posts, and then returned to its starting point and went forward once more assembling the top chords and completing the

span; the top lateral and sway bracing being handled by a swinging boom projecting behind.

Span 5-4 was finished January 30, 1900. The picture, No. 2, taken January 27, shows the span nearly complete, and also shows open water (where the ice had been cut), just upstream of the scows.

A small ice-cutting plant had been bought and put in use for a few days previous to this, and sufficient ice cut and removed to make room for the first movement of the scows about 80 ft. upstream. Up to this date there had been very little extremely cold weather, so that the ice was only 10 or 12 inches thick and the water free from frazil. It was believed that what ice might form in this space before the scows actually started could easily be broken as they were moved. Beginning with the night of January 27, the temperature fell and remained near 0 Fahrenheit for several days. The ice took quickly above the scows, beginning above pier 4 and building out toward the Hull shore. Just above and back of pier 5 the water remained open, and the writer noticed a change in the usual conditions there on February 2, when the scows were first moved. Previous to this there had either been no current at this point or a slight eddy, with chips, etc., drifting upstream; now there was a marked current down stream, showing that the channel farther out must be obstructed, and, of course, with frazil. Moreover, the water was seen to be full of crystals of frazil. This meant more difficulty in moving the falsework than had been anticipated; but if the enormous difficulties could have been foreseen the start would not have been made.

About a mile above the bridge is situate the Chaudiere, or Big Kettle Fall, of the Ottawa River, with perhaps 1,000 feet of open water below, and occasional short stretches of open water, including several lesser falls and rapids for two or three miles above.

This open water is known as an excellent frazil factory. The water used for power in Ottawa and Hull, though taken from near the sides of the river, is often full of frazil, which clogs up the racks and wheels. The water from the central channel of the river, passing through the Chaudiere itself, would naturally contain a still greater quantity of frazil.

These floating ice crystals, which form when the atmospheric temperature in falling approaches or passes below zero Fahrenheit over water which is too much agitated for the surface to freeze solidly, will be called "frazil" in this paper.

As the scows were first to be moved directly upstream or endways, it was expected that they would move quite easily. The writer's diary states that, on February 2:—"Scows were moved only 2 feet. Frazil very deep." This was the first day's experience with frazil. It was found to have other dimensions than depth. On

February 8 there was rain, followed by three rather warm days. By February 11 the river had risen about one foot, and the frazil seemed less dense. On that day, the scows moving broadside across stream advanced 26 feet. This was the record day. Some days only four, five or six feet would be gained. Photo No. 3, taken February 6, shows the scows moved about 80 feet upstream as a result of five days' work.

The method of working, which developed after a few days, was substantially as follows:—

A few men were kept at work night and day with saws, keeping a narrow channel open all around the scows. The surface ice was cut each day for a width of 15 or 20 feet in front of each scow in the direction in which it was desired to move them. This ice was cut with saws, and the blocks pulled out on light skids and drawn away by horses. Large hoes were made by fastening steel snow shovel blades at right angles to long wooden handles. With these the frazil crystals were pulled from in front of the scows upon the uncut ice, and, as soon as a good sized pile was formed, this was scooped up on wooden drag scrapers and drawn away with teams. The frazil would keep rising to the surface for a time, but, generally after a few hours', or, perhaps, half a day's work, there would apparently be about two feet in depth of clear water in front of the scows, about equal to their draft. Orders would then be given to move the falsework, but more frazil would rise and fill this clear space as soon as the scows started.

To some extent the idea still prevails that frazil is heavier than other forms of ice and readily sinks in water. The old theory of the formation of anchor ice by the freezing to each other, and to the bed of a stream, of surface formed needles of ice, has been shown to be incorrect by the work of Dr. Barnes, of McGill University.* The explanation, given by Dr. Barnes, of the formation of anchor ice, as caused by the radiation of heat from the bed of a stream directly through the open water and atmosphere above into space, must, it seems to the writer, be accepted, and result in more clearly distinguishing between two forms of ice that have often been confounded.

The frazil in the Ottawa River showed no tendency to unite by regelation, and none to sink in water. Its tendency to rise is mentioned above, and shows clearly in the annexed photographs.

The crystals of frazil drawn out on the surface ice turned immediately white in the cold air, but remained as separate from each

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*Transactions Royal Society of Canada, Vol. III., June 23, 1897, Vol. V., 1899.

other as particles of coarse salt, the water draining out from between them before it had time to freeze.

A pike pole could be thrust down into the undisturbed beds of frazil more easily than into clear water, as no effort was required to hold it plumb. When released the pole showed no tendency to float or rise up, the friction of the frazil crystals holding it steadily in position. There was just enough cohesion in the masses of tightly packed frazil and, perhaps, adhesion to the bottom of the river, so that with great care about 2 feet in depth at the surface could be scooped off without bringing up the lower layers. The commotion caused by the slight moving of the scows brought new masses of frazil to the surface and above the surface, which, rolling up in front of the scows, quickly formed a barrier which stopped all progress.

The traveler tackles were used to pull the falsework ahead. These were fastened to various scows; some of the lines were run to the engine on the traveler overhead, some to the engines on derrick scow, which was frozen in near pier 5, and two of the fall lines were pulled by teams of horses and the men. The scows would generally resist the combined pull of all the tackles for a few seconds and then, if they moved at all, jump forward a few inches and then move more slowly until the masses of frazil in front brought them to a standstill.

The tackles, which were designed, with a large factor of safety, to lift 10 and 12 tons, were often broken either in the rope, the blocks, or the chains fastening the blocks to the "dead-men" under the ice. So that it was estimated that a pull of 150 tons was exerted to move the scows, often with no other result than a breakage.

It is not believed that the frazil was frozen to the bottom of the scows, as once or twice ropes were pulled under them from end to end.

The mixture of frazil and water, on which the scows floated about as they would if left stranded on a soft mud flat by an outgoing tide, offered a similar if not as great a resistance to pulling the scows over its surface as the soft mud would have done.

It is almost incredible that the amount of frazil first encountered under and around the scows between piers 4 and 5 could have been manufactured and placed there in the few days intervening between January 27 and February 2. It is probable, however, that the part of the channel between piers 3 and 4 had been filled up during January, and that, because of this, the increased current near pier 5 was caused, and this increased current in turn quickly packed the frazil under the surface ice between piers 4 and 5.

The small photographs shown on following pages give some idea

of the difficulties encountered in trying to move the floating falsework over the beds of frazil, and they also show some of the characteristics of frazil. View No. 13 shows the space of open water in advance of one of the intermediate scows when they were moving across stream, and shows the frazil rolling up in front of the scow just as it started to move. View No. 14 shows the frazil rolling up in front of the forward scow just as it started to move, with considerable comparatively clear water still left forward of the frazil. The edge of the solid surface ice, on which the men stood to work, is shown to be irregularly cut, and not so many feet away from the scow at its farther end. The next view, No. 15, shows the frazil completely filling the narrower space between the scow and the solid ice, and nearly filling the wider space in the foreground. View No. 16 is taken immediately after a move of the scows, at a time when the edge of the solid ice happened to be parallel to the scow for the full width of the cutting. Here the frazil is seen completely filling the space between scow and solid ice. In both views 15 and 16 it will be seen that the frazil has been forced up above the level of the top of the solid ice.

View No. 17 shows piles of frazil and a workman standing away from the ropes while the strain of pulling the scows forward is on them. View No. 18 shows piles of frazil being shovelled by workmen. It is quite evident that the crystals are not frozen together.

The new position between piers 3 and 4 was reached in the afternoon of February 23rd, but, in the meantime, one end panel of Howe Truss bottom chord had been pulled off and all the scows moved from the correct position under the trusses. To have repaired the damage and replaced the scows in their former position, necessitating moving each one through the frazil, would have taken three or four days' time, or to about February 27th. Counting from Feb'y. 2nd to 27th, including the Sundays and nights to 10 p.m., on which work was done, this would be equivalent to at least twenty-six full days' work for moving the falsework through the frazil with a force of about 50 men and 12 to 16 horses. The actual cost was about $3,500.00, including the expense of moving three of the scows 15 feet each through the frazil to new positions under the truss.

Before the scows had reached their new position the plans for carrying on the work had been changed. The Ottawa River occasionally breaks up at this point by the middle of March. The shoes on pier 4, as described in Mr. Shearwood's paper, had been designed with two pins each—the lower pin for the end of the cantilever anchor arm, the upper one being the end pin of span 4-5. This arrangement utilized the weight of half the latter span as anchorage for the cantilever and saved in the pier anchorage; but it made it dangerous to start the erection of span 3-4 so late in the season.

when a sudden rise in the river might, with floating falsework, have wrecked two spans. If the work could have been started two months earlier than it was, the scows would have been moved the first time through clear water in a few hours. Or, if there had been only surface ice to contend with, this could have been cut and the scows moved in three or four days. Spans 3-4 could then have been finished and the floating falsework taken across to span 1-2 for work there during the spring high water, as originally planned. It was now decided to rearrange the falsework for span 1-2, and to erect part of the material intended for span 3-4, working leisurely until the river broke up, and then move the falsework and partly erected span across the river. In order that when erecting span 1-2 the scows should be as far as possible from the rocky shore in front of and just below pier 1, three of them were moved, one Howe truss panel length, or 15 feet 5¼ inches toward pier 3, and vertical posts placed under the top falsework bents, extending down to the 1-beams on the scows. This is shown on the profile view, Plate 16, with falsework reversed from its position at 3-4. Even with this new arrangement (which had been intended for span 1-2), it had been found necessary to blast considerable rock to clear the inshore scow. This work had been laid out and finished in January. Without working under water, the rock could only be taken down to elevation 94. Hence, with the scows drawing 5 feet the one next to pier 1 would still ground, with water at elevation 99.

The top falsework was now cut down 5 feet shorter, and 5 feet of blocking put under the Howe trusses on the scows. A "Jacking rig" had been added to the Howe trusses, which is shown in Plate 16.

By means of this, aided by pumping water into and then out of each scow, the trusses had been jacked up for this blocking, just as it was intended later to jack them down. About 3 feet were cut off the Howe truss chords at the end next to pier 3, and the falsework moved in that direction so as to clear the double shoe on pier 4. The erection of what should have been span 4-3 began March 11 and progressed until April 2, when as much material was assembled as it was thought safe to bring across the river.

It happened that the break up in the Ottawa River came late in 1900. The photo No. 4 taken April 2, shows the partly erected span and solid ice and piles of frazil still around the scows, and it is possible that span 4-3 might have been erected without accident. Had such an attempt, however, been made in 1898 or 1900 it would have certainly resulted in a wreck. In 1898, the rise began March 10th, and culminated March 19th.

The erection men were now put at work on the trestle approaches. The river began rising soon after this, and the ice went out April 15. On the 19th the derrick scow was towed across the

river, and, on the 21st, three tugs took the floating falsework across. This was first pulled down stream and turned end for end, then towed upstream and across to just above its new position, and then dropped into place between piers 1 and 2. The photo No. 5 shows the falsework, with part of span and traveler, as it crossed the river. Photo-No. 6 shows the falsework between piers 1 and 2, with the metal work above the top of pier 2.

The water was now at elevation 101. One of the 12 ton stiff-leg derricks had been set up just back of pier 1. This lifted the material off the derrick scow, as the latter brought it over from Hull, and landed it on the track on span 1-2. This span was connected May 21, when the work of jacking down began with a small force. About 10 inches of blocking at a time was taken out from over each scow, water being pumped into the scow to help the jacks and then pumped out again.

The river rose this year only to 105.5 (the extreme high water mark is 115.4), and was again down to elevation 101 when the span was landed on the masonry, May 29th. The span is shown resting on the piers in photo No. 7, taken May 31st.

In the meantime there had been exciting episodes connected with the great Ottawa-Hull fire. On the afternoon of the fire square piles of blazing boards came floating down the river, but the upstream wind held them back, so that what fire remained when they reached the scows was easily extinguished by throwing on water from buckets. There was a great deal of debris in the river after the fire, and one long and heavy boom floated down and tied itself all round the scows one Sunday morning, when it was difficult to get tugs to pull it away.

The falsework was held in place by chains and anchors and by pier harnesses made of 14 inches by 14 inches upright timbers fastened to the piers by a system of rods and timbers running completely around the stone work. These are shown in photo No. 4. The uprights were spaced about 27 feet apart in the clear so that the Howe truss chords could move up and down between them. The chords were kept wedged against the harness timbers, the watchman loosening and re-adjusting the wedges occasionally as the water level changed.

On June 2 the unloaded falsework was moved back to between piers 3 and 4. The traveler remained at work on the cantilever erection, finishing this to centre of suspended span on July 10th. The progress of erection to June 23rd is shown in photo No. 8. On July 11th the work of takihg down the traveler began. The top trusses were lowered to the derrick scows *complete*, and after that the tower in pieces. This work was finished at noon of the 16th, in 4½ working days.

The erection of the traveler over pier 4, at the end of span 5-4, began on the 17th and was practically finished on the 21st. On Monday, July 23rd, engines and tackles were raised, and erection of span started in the afternoon of the 24th. Photo No. 9 shows span 5-4 finished and cantilever erection started August 30th, 1900.

There is nothing of special interest to describe in the rest of the work except the method of making the final connection in the centre of the cantilever span. No wedges had been arranged in the trusses, as it was known that the floating falsework would be on hand in case of need. There were adjustments for length in the chords, at U13 for tension and L13 for compression.* Shim plates were inserted at these points to give the length of half spans, which, it was thought, would enable connections to be made at what would be the average temperature at the expected time of connecting, about October 1. The temperature at this time was lower, and the central opening was found to be about one inch greater than had been calculated on. On Saturday, October 6, the floating falsework, with enough water pumped into the scows to give about 4 feet draft, was brought out under the suspended span. Blocking was inserted between the falsework and the panel points of the span, and the water partly pumped out of the scows on Saturday night. On Sunday morning the pumping was finished and the weight of the span lifted. The adjusting yokes at U13 were loosened, and the shim plates removed at both U13 and L13. The half spans were then jacked together and the last pins driven about 11 a.m., October 7th. Water was then again pumped into the scows, and on Monday, October 8th, they were towed to moorings on the Hull shore, where the work of dismantling the top falsework and Howe trusses began, the material being piled up on the scows and taken on board the latter back to the shop at Lachine.

The progress of the cantilever erection to September 18th is shown in photo No. 10. Photo No. 11 shows the falsework under the suspended span while the last pin was being driven, October 7th.

Concerning the cost of the work, a few items only can be given.

The falsework scows cost about $2,400 each, and were readily sold at two-thirds their cost. The cost of moving them through the frazil has already been given, $3,500. This was an unusual and unexpected expense. The Howe trusses were easily taken apart, and the material in them was of considerable value for other work. A suitable falsework span, at least 200 feet long, for use between piers 1 and 2, and which would, necessarily, have been erected on shore and floated into position, would probably have cost nearly as much

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*See Plate 15 Shearwood's paper.

as the falsework used for the three spans. The floating falsework, if it could have been moved before the appearance of frazil, would at least have saved the cost of the heavy bents that would otherwise have been required for spans 3-4 and 4-5.

The superstructure was designed in the office of the Dominion Bridge Co., Ltd., Phelps Johnson and G. H. Duggan, Members Can. Soc. C.E., respectively Manager and Chief Engineer, and F. P. Shearwood, Asso. Mem., Assistant Engineer, in charge of computations and drawings. Jas. Finley was in direct charge of men employed on erection.

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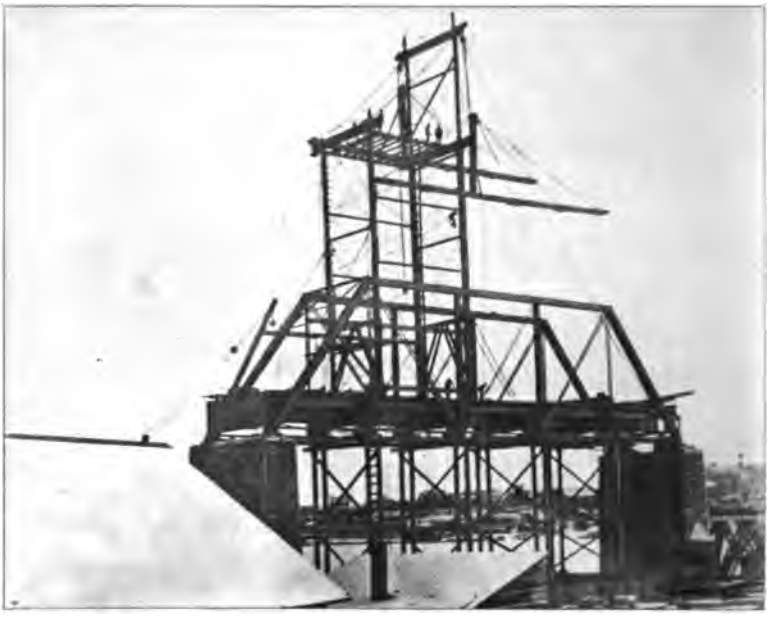


1 Raising Top Trusses of Traveler
January 10, 1900

Erecting Span 4-5 and Cutting Ice Preparatory to Moving Scows
2 January 29, 1901

3 Scows Moved Up Stream Clear of Piers
February 6, 1900

Span Partly Erected, Forward of Correct Position to Clear
4 Double Shoe on Pier 5
April 2, 1900

5 Crossing Ottawa River
April 21, 1900

6 Falsework in New Position and Erection of Span 1-2 in Progress
April 23, 1900

7 Span 1-2 Resting on Piers
May 31, 1900

Erecting First Panel of Suspended Span at End of Cantilever Arm
8 June 23, 1900

9 General View from Nepean Point, Showing Floating Falsework at Span 3-4
August 30, 1900

10 Ready for Central Panels of Suspended Span
September 18, 1900

11 Driving Last Pin
Oct. 7, 1900

Finished Bridge
Dec., 1900

13. Intermediate Scow Moving.

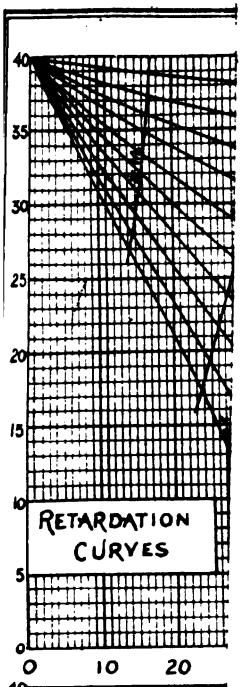
14. Forward Scow Moving.

15. Forward Scow just after Moving. Frazil Floated up above Water.

16. Forward Scow just after Moving.

17. Scows Moving. Piles of Frazil.

18. Forward Scow after Moving. Piles of Loose Frazil being Shovelled.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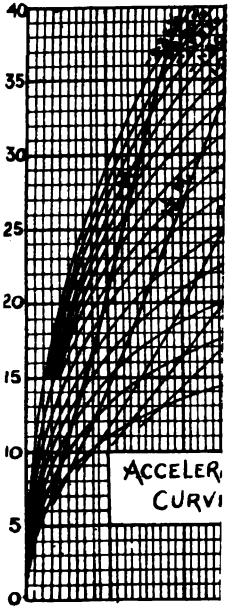
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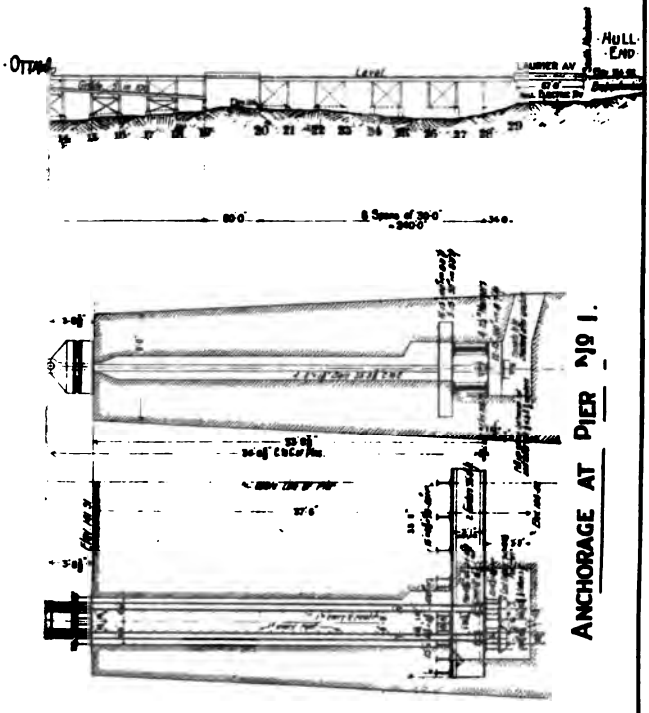


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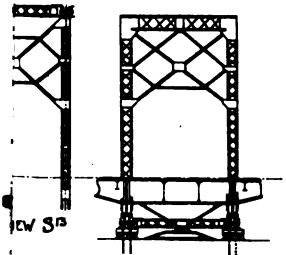

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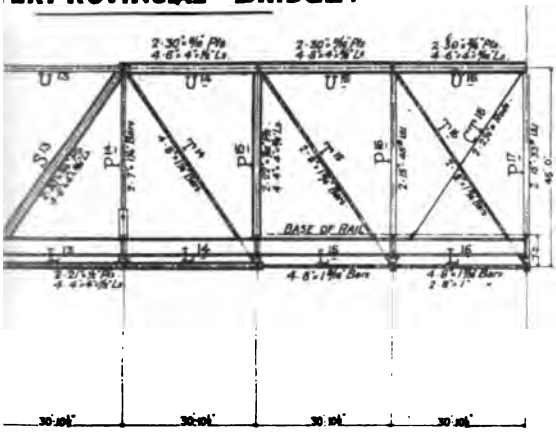


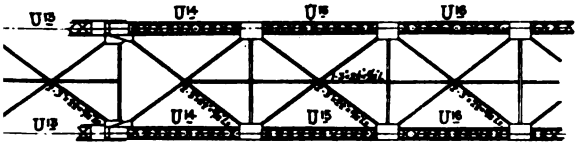

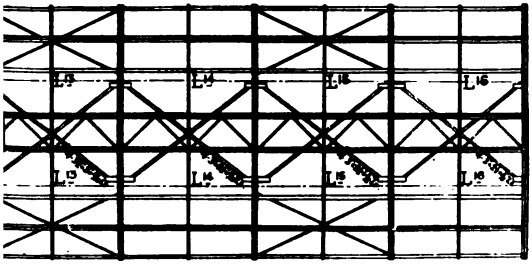

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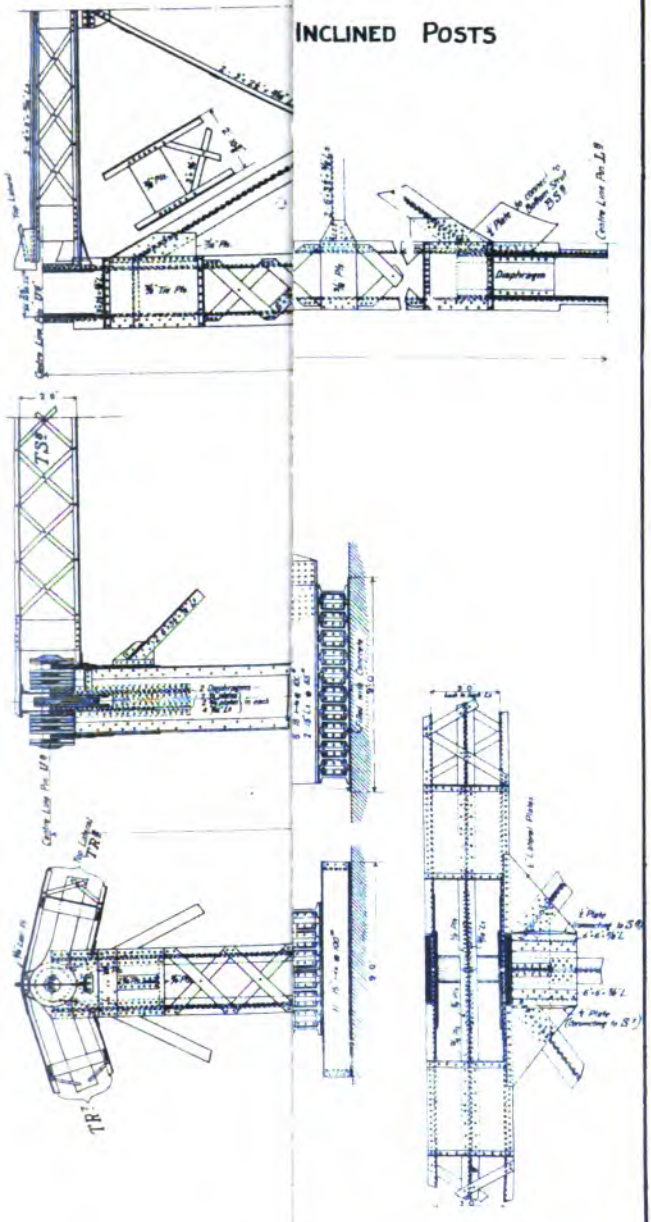

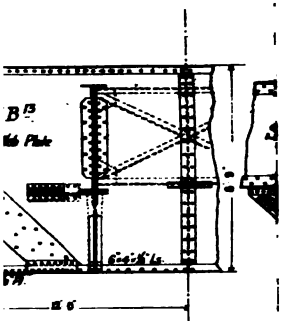

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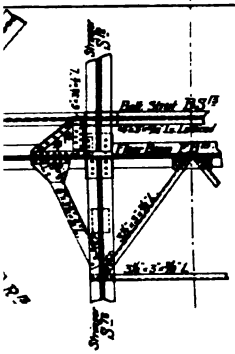

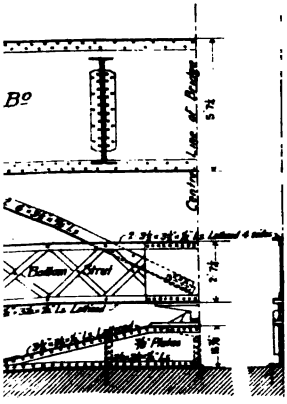

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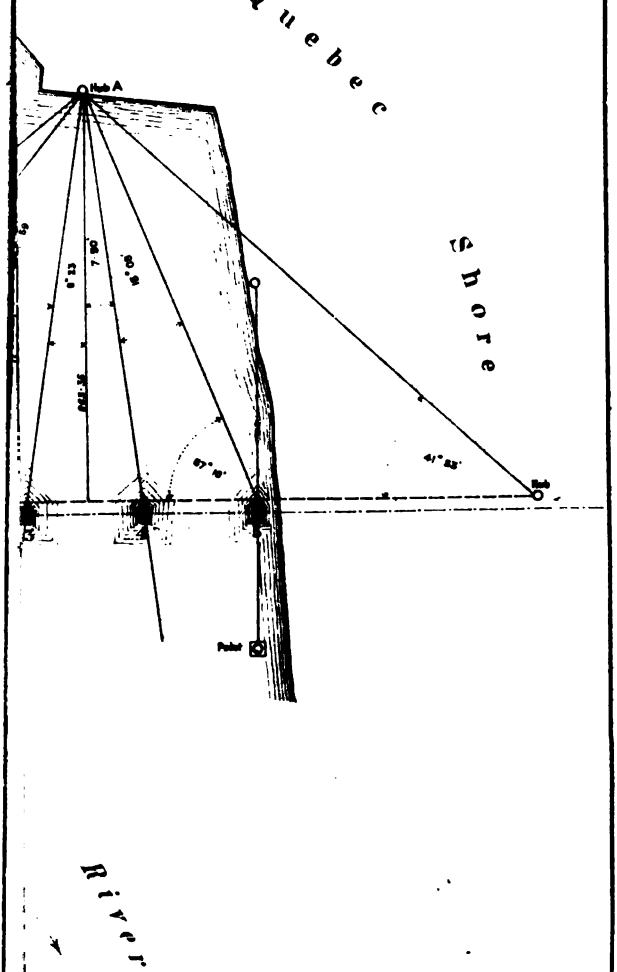


· INTERPROVINCIAL · BRIDGE ·

✣ OTTAWA ✣

PONTIAC · PACIFIC · JUNCTION

· OTTAWA · & · GATINEAU · RAILWAYS ·

General Plan and Profile
and
Isometrical Drawings of Piers.

L · BRIDGE ·

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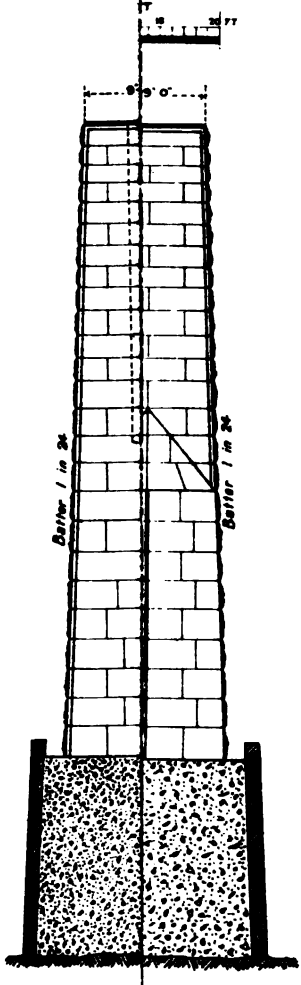


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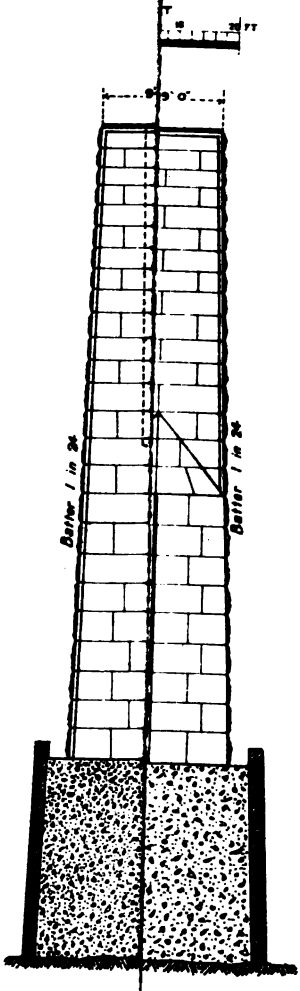


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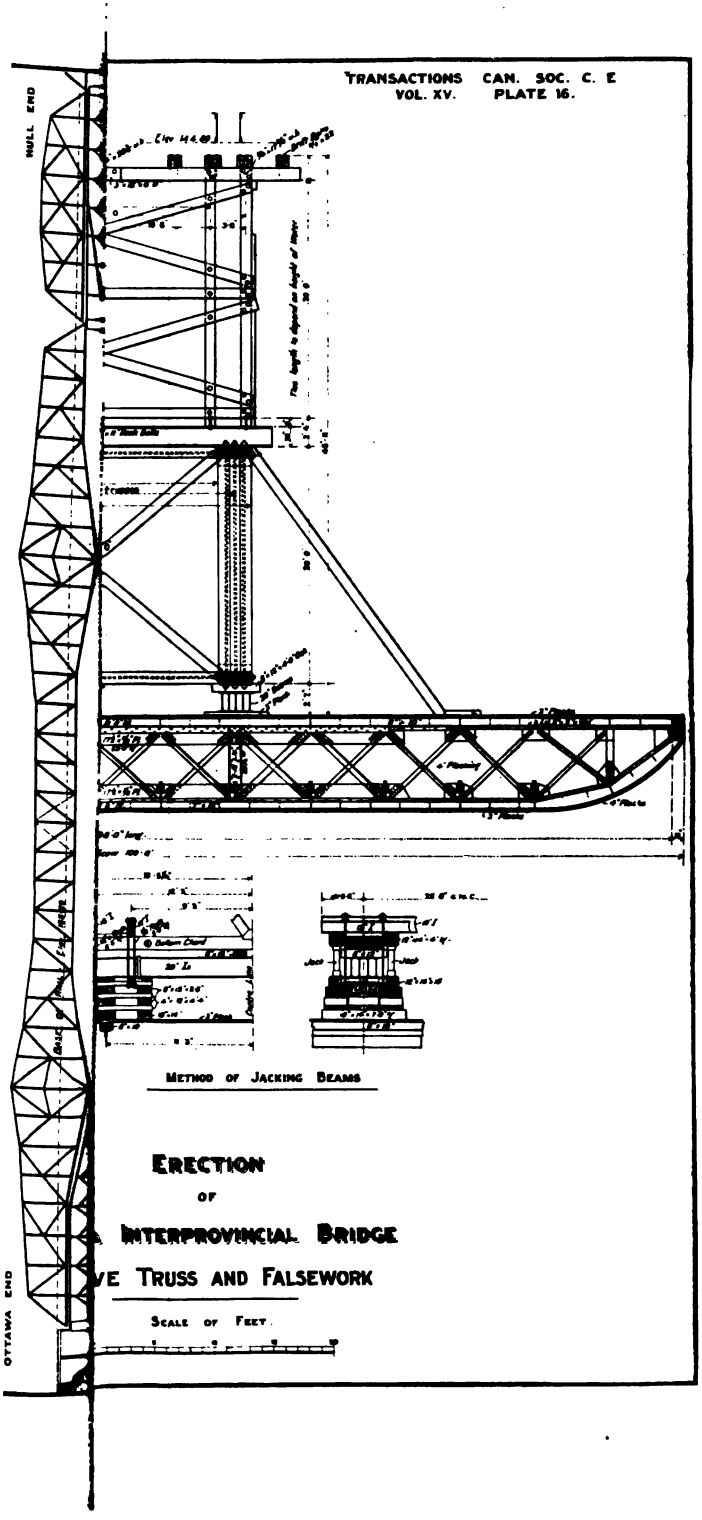
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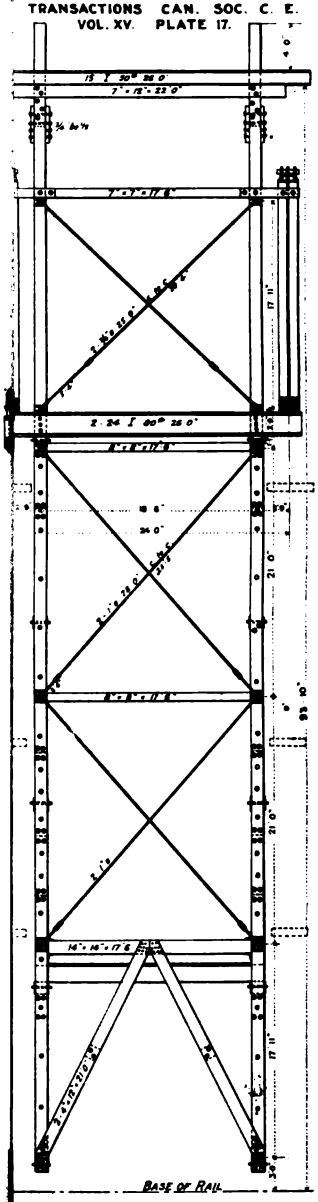


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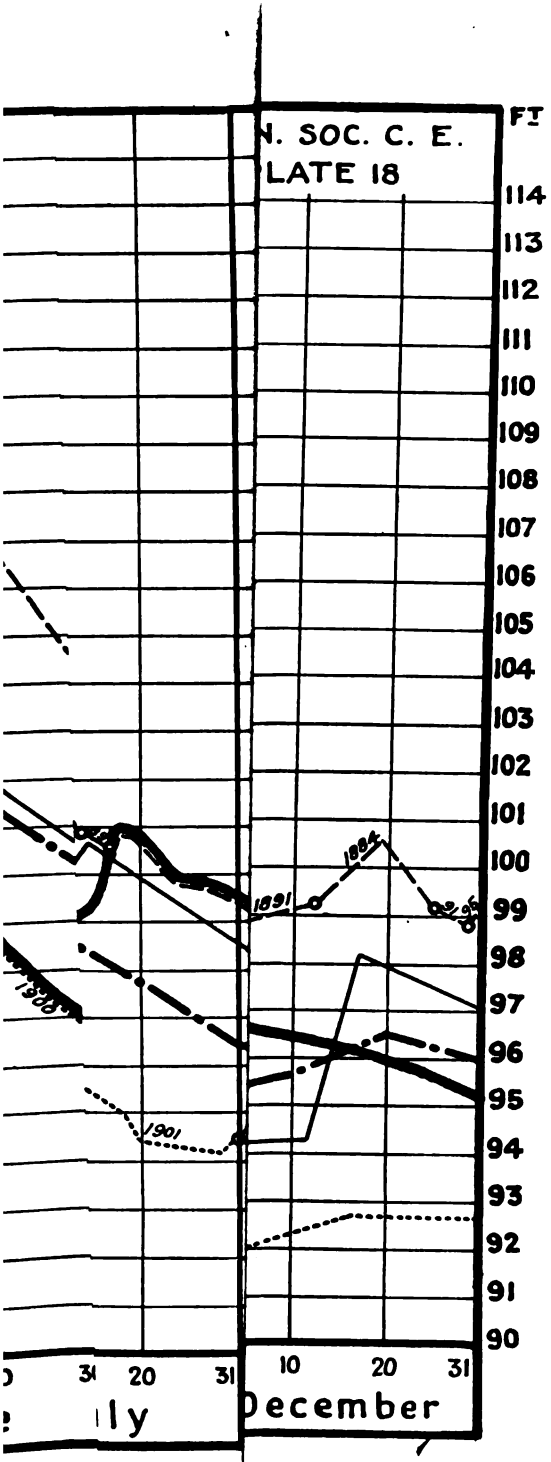
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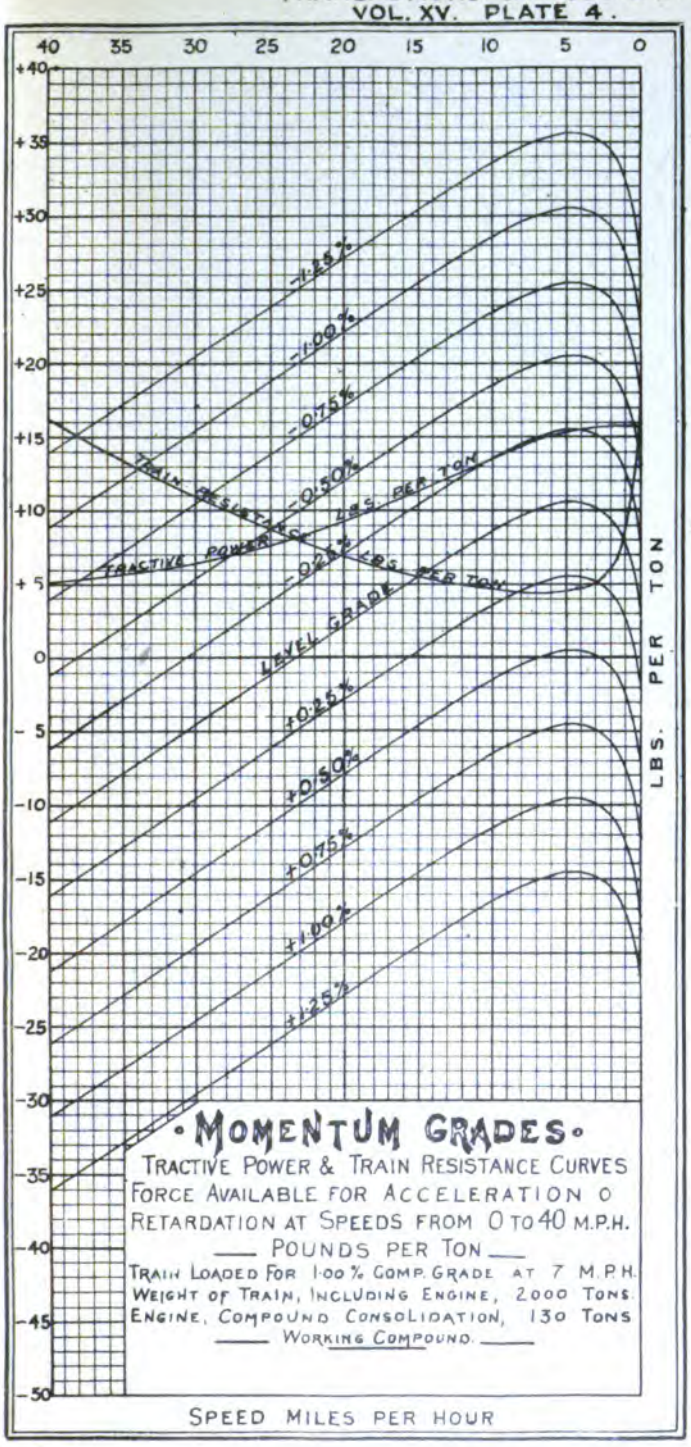
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TRAIN RESISTANCE LBS. PER TON
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LBS. PER TON
·MOMENTUM GRADES·
TRACTIVE POWER & TRAIN RESISTANCE CURVES
FORCE AVAILABLE FOR ACCELERATION O
RETARDATION AT SPEEDS FROM 0 TO 40 M.P.H.
POUNDS PER TON
TRAIN LOADED FOR 1.00% COMP. GRADE AT 7 M.P.H.
WEIGHT OF TRAIN, INCLUDING ENGINE, 2000 TONS.
ENGINE, COMPOUND CONSOLIDATION, 130 TONS
WORKING COMPOUND.
SPEED MILES PER HOUR

1902, May 29 Engineering Library

# Canadian Society of Civil Engineers

# Index of Transactions

Vols. I—XIV

Montreal 1911

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OF

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